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Introduction to
HIGHWAY ENGINEERING

A TEXTBOOK FOR
STUDENTS OF CIVIL ENGINEERING

BY

JOHN H. BATEMAN
PROFESSOR OF CIVIL ENGINEERING
LOUISIANA STATE UNIVERSITY

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PREFACE

This book was written to serve as a textbook for students of civil engineering in an introductory course in highway engineering. Emphasis has been placed on fundamental principles and processes in the practice of highway engineering. It has been assumed that the student will receive instruction in basic courses in surveying, drafting, mechanics of materials, chemistry, materials of construction, and structural design, and therefore detailed descriptions of laboratory procedures, test methods, manufacture of materials, the proportioning of concrete mixtures, surveying operations, and the design of bridges have been avoided.

In the first chapter the student is given a brief historical background and concise descriptions of highway systems and the highway structure itself. The following five chapters deal with subjects which should precede the study of highway surfaces. Details of the design, construction, and maintenance of various highway types then are described. These subjects serve as a logical background for a discussion of surveys, plans, specifications, economics, planning, financing, and administration

The changes in the Fourth Edition comprise a revision of much of the subject matter of the previous edition, the introduction of a considerable amount of new material, and a slight rearrangement of the chapters. The material on roadside improvement, properties of bituminous materials, and the stabilization of subgrades with soil-cement mixtures has been expanded. The chapter on sheet asphalt and asphaltic concrete pavements has been revised to present the subjects of proportioning and control of asphalt paving mixtures in a more effective manner. The subject matter dealing with the structural design of pavements has been greatly expanded to include the excellent contributions to our knowledge by Goldbeck, Hubbard, and Field, Westergaard, Bradbury, E. F. Kelley, and many others. The material on the theory of the structural design of rigid pavements of concrete has been placed in a separate chapter immediately before the chapter on concrete pavements.

The author has received helpful suggestions from many teachers of highway engineering, from practicing engineers, and from former students who have critically reviewed the preceding edition. He particularly acknowledges his indebtedness to Professor R. L. Morrison of the University of Michigan for criticism and suggestions relative to the arrangement of the chapters in the Fourth Edition.

JOHN H. BATEMAN

BATON ROUGE, LOUISIANA,
May 1, 1942

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CHAPTER I

DEVELOPMENT OF HIGHWAYS

The field of highway engineering embraces the following divisions:

1. Design.
2. Construction.
3. Maintenance.
4. Economics.
5. Administration, financing, and planning.

Before proceeding to a study of these various phases of highway engineering, the student should have at least a brief knowledge of the development of highway transportation, of the development of highway systems, and of the elements of the highway structure itself.

EARLY HIGHWAYS

We know very little about the first roads, but we do know that they date back prior to the so-called historical period. Traces of old roads found in Central America and northern South America are believed to represent roads which were constructed by Incan peoples. One of the oldest roads of which there are traces today is on the island of Crete; this road, which was surfaced with stone, probably was constructed before 1500 B.C.

Early Roman Highways. During the period from approximately 400 B.C. to A.D. 200, the Romans constructed a vast system of roads, embracing a total length of more than 75,000 miles, which extended over their whole Empire. Twenty-nine highways radiated from Rome and connected with numerous other roads. Many of these roads were surfaced with several layers of stone, in many locations so thick and of such large size that traces of them may be found today. After the decline of the Empire, road building did not flourish again until the latter part of the eighteenth century, when it was renewed in England and France, and to some extent in the United States.

Early French Highways. The roads in France which had been built by the Romans fell into disuse, and road conditions were bad until nearly the beginning of the nineteenth century. The early development of highways in France was due in a large measure to Tresaguet (1764), one of the most famous forerunners of modern highway engineers. Tresaguet developed numerous methods of construction which were meritorious. Napoleon recognized the value of highways, and the present national road system of France had its beginnings during his régime from 1804 to 1814. It was about this time that the French Department of Roads and Bridges was established.

Early British Highways. Road conditions in England following the decline of the Roman Empire were as bad as on the Continent, and a long period of time elapsed before much attention was given to them. A progressive step was made in 1555 when an Act of Parliament placed the roads under the charge of surveyors. The first attempts to improve road conditions were made by Macadam (1783), who, like Tresaguet in France, contributed a great deal to the art of road building. Macadam developed a method of constructing broken stone roads, and his name still is associated with roads built of this material.

HIGHWAY DEVELOPMENT IN THE UNITED STATES

The first highways in the United States served local purposes, only, and the development of important cross-country routes began when exploration westward started. The Wilderness Road to Kentucky was laid out by Daniel Boone in 1775, and other overland routes followed, including many roads of great historic interest, such as the Natchez Trace from Natchez to Nashville, the Santa Fe Trail, the Oregon Trail, and the Overland Trail.

The first participation of the national government occurred in 1802 when Congress passed an act for the improvement of a road, which became known as the National Pike, from the Potomac River to the Ohio River. In 1805 the improvement of this road westerly from Cumberland, Maryland, was authorized and provision was made for surfacing. The first contract for construction was awarded in 1811, and by 1821 the road had been completed as far as Wheeling, West Virginia. The first appropriation for this highway west of the Ohio River was made in 1825; the final appropriation by the national government was made in 1838, and by this time the section in Ohio had been completed. Some work was done on this road in Indiana and Illinois, but very little surfacing was placed. By this time the construction of railroads had begun and provided the opportunity of developing the interior of the country. The importance of roads as through routes of travel diminished, and the national government ceased its participation in the improvement of highways. Local traffic was the most important function of roads and streets until the development of the motor vehicle.

Early Types of Surfaces. Early highways in the United States generally had natural earth surfaces. Numerous city streets were paved with cobblestones, and the use of wood blocks soon followed, round cedar blocks being used extensively. The high cost of surfacing led to the construction of large numbers of toll roads which were both privately and publicly operated and which were surfaced with gravel, logs, or planks. The influence of Macadam and Telford in England extended to this country and led to the use of broken stone to some extent. The invention of the power stone crusher in 1858 and of the steam roller in 1859 greatly increased the use of broken stone by lowering the cost of its production. These surfaces were satisfactory for horse-drawn traffic because the contact of the horses' shoes and of steel tires on the stone provided a small amount of dust which, when moistened with water from rains, acted as a natural cement to fill the void spaces in the surfaces and hold the individual stone fragments together. By the close of the nineteenth century large mileages of broken-stone surfaces had been built on municipal streets and on rural highways. Gravel was used in large quantities in locations where it was economically available. The increase of traffic on city streets and on some adjacent rural highways necessitated more durable wearing surfaces. Cobblestones and untreated wood blocks were not satisfactory. Stone blocks were used in those locations where sandstone and granite, whose natural cleavage planes permitted the manufacture of somewhat smooth blocks at reasonable cost, were economically available. One of the first of modern surfacings was rock asphalt which was imported from Europe and was used in cities along the Atlantic seaboard and on the Gulf coast. The first brick pavement was constructed in Charleston, West Virginia, in 1871, and sheet asphalt was first used in 1879 on Pennsylvania Avenue in Washington, D. C. Bellefontaine, Ohio, is credited with having the first street pavement of Portland cement concrete. Wayne County, Michigan, constructed the first rural road of concrete in 1909.

Beginning of Motor-Vehicle Travel. The year 1904 marked the beginning of motor-vehicle travel. Motor vehicles had been developed prior to this time, but their use had been restricted to urban streets. Mechanical improvements developed rapidly, and about 1904 or 1905 motor vehicles in large numbers commenced to use rural highways and, because of their economy and speed of operation, made highway development a necessity in modern life.

The motor vehicle immediately confronted the country with a tremendous highway problem. Many municipalities and rural sections found that their broken-stone streets and roads, which were admirably adapted to horse-drawn vehicles, were being destroyed rapidly by this new power vehicle. The utilization of bituminous surfaces on broken-stone roads and streets was perhaps the next most significant development in connection with highway travel. In the eastern states, particularly, very large mileages of roads and streets were treated with bituminous materials which were the means of protecting the large expenditures which already had been made for travel purposes.

The development of motor-vehicle transportation presented another problem in highway improvement of far-reaching importance, namely, the controversy of farm-to-market highways *vs.* through highways. This problem affected those states which were largely agricultural. The extensive railroad construction in the United States had opened up large areas to agricultural development and provided means for transporting farm products to markets. The improvement of local roads to serve farm travel was urged as a means of further agricultural development. Along with this demand arose the problem of improving the main traveled routes between the principal centers of population. Farm-to-market roads did not always serve through-traffic, and in many areas the principal highways suffered from lack of improvement. This condition was an important factor in the development of the principle of federal aid for highways of interstate importance

Effect of the World War. The development of highway travel after the entry of the United States into the World War brought many new problems into prominence, of which one of the most important was the subject of loads. The transportation of commodities principally from the interior to the Atlantic seaboard immediately demonstrated the inability of existing highways to withstand the heavy loads which were being hauled by motor truck. Load restrictions on existing highways and the design of road surfaces on new and reconstructed highways became of paramount importance. The necessity of bridge-building programs was speedily recognized. New problems relating to the construction and financing of interstate highways required solution. Highway improvement thus assumed a position of vast importance in the industrial, agricultural, and social life of the country.

State Aid Development. During the early stages of our historical development the improvement of highways was largely under the control of local governmental units with some national participation that already has been described. The agricultural development, which was made possible by the country's railroad facilities, soon caused the participation of state agencies in highway improvement. The first state-aid law was passed by New Jersey in 1891, and by 1900 the states of Vermont, New York, Massachusetts, Maryland, Connecticut, and California had passed similar legislation. The rapid use of the motor vehicle soon afterwards caused other states to participate in road building. In many states, this participation was affected to a large degree by the farm-to-market *vs.* through-highway problem, and much of the actual improvement was scattered and disconnected.

Federal Participation in Highway Building. The first activities of the national government in highway improvement consisted of direct participation in construction and have been described. These activities ceased with the work on the National Pike about 1840, and it was not until 1893 that the national government again became actively interested. In that year the U. S. Office of Road Inquiry was created by an Act of Congress. This office was placed under the Department of Agriculture, and for a period of 23 years its work was entirely educational. Its name was later changed to the Bureau of Public Roads. On July 1, 1939, the functions of the Bureau were transferred to the U. S. Public Roads Administration, a division of the Federal Works Agency. In 1916 Congress passed the Federal Aid Road Act, which was the direct result in a large degree of the farm-to-market *vs.* through-highways controversy. An appropriation of $75,000,000, to be expended in five years, was provided and was apportioned among the states in proportion to their area, population, and postal route mileage. The federal government's participation was limited to 50 per cent of the cost of construction and a maximum of $10,000 per mile. The postal route feature has been modified, and the maximum limit has since been changed. The Federal Aid Road Act required each state to have a state highway department and drew the attention of the states to the necessity of planned highway systems.

About this time strong efforts were made to establish a national highway system to be financed, constructed, and maintained by the national government. These efforts did not meet with success, which was due in large measure to the popularity of the federal aid plan among the various states. The passage, by Congress, of the Federal Highway Act, in 1921, overcame many of the objections to the federal aid plan raised by supporters of the national highway plan. The Federal Highway Act of 1921 provided for the establishment of a federal highway system and made its maintenance mandatory upon the various states.

Large appropriations for road purposes were made by Congress in connection with emergency measures to relieve the conditions caused by the financial depression which began in 1929 and reached its low level in 1933. Portions of these appropriations were allocated to primary highways, to city streets on state and interstate routes, and to secondary highways which connected with state highways.

HIGHWAY SYSTEMS

The public highway systems of the United States include:

1. The national highway system.
2. State highway systems
3. County highway systems
4. Town, or township, roads
5. Municipal streets.

National Highway System. There is no national system of highways in the United States which is constructed and maintained directly by the federal government. The federal government, as previously stated, participates in the construction of highways of interstate importance by granting federal aid to the various states. As a result of a cooperative arrangement between the American Association of State Highway Officials and the U. S. Department of Agriculture, a comprehensive system of interstate highways was selected by a Joint Board on Interstate Highways in 1925, and a uniform system of numbering and designating these routes for the convenience of the public was recommended. Since that time this system of interstate highways has been expanded and generally has been adequately marked with marker signs in the shape of a shield, bearing the name of the state, the letters "U S," and the number of the route. Even numbers are used to designate easterly and westerly routes and odd numbers to designate northerly and southerly routes. This system of "U S" highways, which is constructed and maintained by the various states with the help of the federal government, has thus provided the equivalent of a national highway system.

The practical completion of surfacing on the principal interstate highways by 1938, the large increase in highway traffic, and the requirements of national defense, led to the recommendation of an interregional highway system comprising about 30,000 miles of highways and providing for the betterment of existing routes to carry swiftly moving traffic between all of the major population centers of the country. This program, involving the modernization of the federal-aid highway system, including the elimination of railroad and many highway grade crossings and the improvement of secondary and feeder roads, will constitute the next stage in highway development in the United States.

State Highway Systems. Soon after the advent of the motor vehicle, many states recognized the fact that certain highways were of statewide importance and, by legislative enactment, created state highway systems. In most of the states, the state highway system embraced roads of interstate importance and those which connected the main cities and villages. Thus these systems have become well established and future developments will consist of alterations of existing routes to facilitate travel and additions made necessary by the development of specific areas.

Local Highway Systems. Until the advent of the motor vehicle, rural highways were largely local in importance and their care was entrusted to local officials. In many sections of the country, particularly in the Middle West, roads were under township control; in New England they were under the control of towns; and in other sections of the country, particularly in the South, they were under county[1] jurisdiction. In many states having township units, especially in the Middle West, as highway transportation developed, county highway systems were established to care for the more heavily traveled roads, leaving only the local farm roads in the control of the township units. Unless a township embraces a large area and has large financial resources, it is too small a unit to administer highway improvement and maintenance efficiently. Its purchasing power usually is limited, and it cannot develop a continuing organization with the necessary equipment to do effective work. Some states have eliminated the township unit in highway administration and placed all local roads under the care of county units. North Carolina and Virginia have placed all roads under the care of their state highway departments.

Municipal Streets. The administrative control of street improvements is generally vested in a street department, engineering department, or department of public works. Municipal streets form a very important part of the entire highway transportation system, and most states provide for cooperative financing of street thoroughfares which form a part of state or county highway systems.

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[1] In Louisiana, the *parish* corresponds to the county unit in other states.

THE HIGHWAY STRUCTURE

The highway structure embraces the entire right-of-way, which is the area occupied by the highway between the boundary lines with private property on each side, and consists of:

I. The subgrade, which is the soil below the roadway and below the shoulders.
II. The roadway, or traveled portion of the highway; it may consist of one or more courses of road-surfacing material and may include a foundation. It is further described as consisting of a number of traffic lanes, each sufficiently wide to accommodate the vehicles which use it and to provide safe clearance between vehicles in adjacent lanes.
III. The shoulders, which are the two strips of earth or other material, one on either side of the roadway, between the roadway and the inside boundary of the ditch or gutter.
IV. The roadside, which is the area between each boundary line and the edge of the adjacent ditch or gutter; if there are no ditches or gutters, no visible boundary exists between the shoulders and the roadside.
V. The drainage system, which includes ditches, gutters, drains, spillways culverts, and bridges.
VI. Miscellaneous structures such as guard railing, sidewalks, retaining walls, lighting systems, grade-crossing treatments, and signs.

Classification of Roadway Surfaces. We may classify the various wearing surfaces or pavements into three principal groups as follows:

I. Low-type surfaces:
    1. Natural earth.
    2. Sand-clay.
    3. Gravel.
    4. Stabilized soil.
    5. Traffic-bound macadam.
    6. Some special surfaces including iron ore, shell, limerock, etc.
II. Intermediate-type surfaces:
    1. Traffic-bound and water-bound macadam with bituminous surface treatment.
    2. Bituminous road mixes with adequate foundations.
    3. "Low-cost" bituminous plant mixes with adequate foundations.
    4. Cement-bound macadam.
    5. Bituminous macadam.
III. High-type surfaces:
    1. Concrete.
    2. Bituminous concrete, sheet asphalt.
    3. Rock asphalt.
    4. Block pavements.

The foregoing classification is subject to differences of opinion. Bituminous macadam, for example, generally is described as an intermediate type, but in many cases may be classified, quite properly, as a high-type surface; thin rock asphalt surfaces on gravel and broken stone foundations may be, and frequently are, described as of intermediate type.

Flexible and Rigid Types of Pavements. Roadways may be further classified as *rigid* and *flexible.* Pavements of Portland cement concrete or those in which Portland cement concrete bases or foundations are used are known as rigid pavements. Gravel, the various types of macadam, and asphaltic or bituminous wearing courses supported on gravel, macadam, or bituminous foundations are known as flexible pavements or roadways.

CHAPTER II

HIGHWAY SUBGRADE SOILS

Highway wearing surfaces and pavements serve two purposes—they provide resistance to the wear of traffic, and they transmit the weight (including impact) of vehicles to the underlying subgrade soil. In addition to the forces imposed by traffic, highway wearing courses and pavements are subjected to forces resulting from movements in the subgrade soil. The highway engineer, therefore, is interested not only in the effect of the wearing surface on the subgrade but also in the effect of the subgrade on the wearing surface. Although our knowledge of soils is by no means complete, a large amount of information, which is helpful in the design, construction, and maintenance of highways and streets, is available.

Constituents of Soils. Soils are formed by the disintegration and decomposition of rocks through the action of temperature changes, water, ice, frost, and plant and animal life The upper layer of fine-grained material, which supports plant life, contains humus resulting from the decay of vegetation and is known as the soil proper. Subsoil is the underlying soil and is less weathered and usually contains little or no vegetable matter Soils are composed of granular mineral material, organic matter, and various inorganic chemical compounds. The granular mineral material is classified into five constituents called gravel, sand, silt, clay, and colloids, which are defined, rather arbitrarily, according to their grain size or diameter as follows:

Gravel	over 2.00 mm
Sand	2.00 to 0.05 mm
Silt	0.05 to 0.005 mm
Clay	below 0.005 mm
Colloids	below 0.001 mm

Colloids are extremely small particles; inorganic colloids are not a separate mineral but are clay in a finely divided form and are gluey in character. Soils also contain organic colloids which are gelatinous in character. Mixtures of sand, silt, and clay with humus are known as loams. Muck is the soft silt, mixed with organic matter, which is found in swamps or is deposited by overflowing rivers. Peat is soil formed by the decomposition of vegetable matter under water. The inorganic chemical compounds of greatest importance with reference to their effects on subgrade soils are lime and magnesium compounds which tend to flocculate fine-grained soils, and sodium and potassium compounds which tend to deflocculate or disperse fine-grained soils.

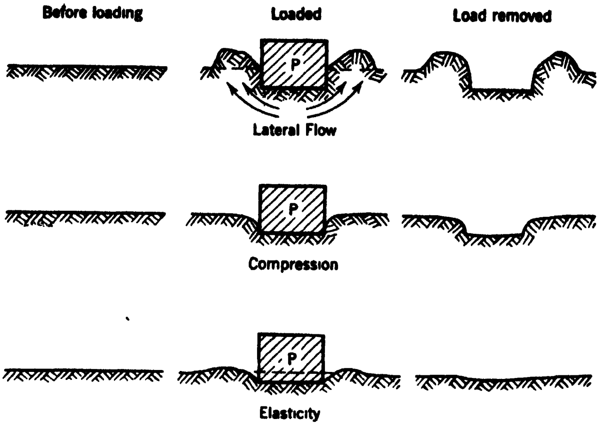
Water is a very important constituent of soils and may be present as free water, which is in the ordinary liquid form, or as adsorbed water, which exists as a film surrounding and attracted to soil particles by adhesive force. Water enters soils as the result of the gravitational flow of rainwater or water resulting from the melting of ice and snow, or as the result of capillary flow from an adjacent source of supply. Capillary flow can occur only when the soil pores are small enough to create surface tension. Free gravitational water may be removed from soils by drainage; water can be removed from capillary soils by drainage only when the quantity of water present is in excess of that retained by capillary force

PROPERTIES OF SOILS

The properties of soils of greatest importance to the highway engineer are density, supporting power, permeability, compressibility, elasticity, and capillarity.

Density. The density or degree of consolidation of a subgrade soil exerts an important influence on the supporting power and on the water-bearing capacity of the soil. The density of a soil may be expressed in terms of its porosity or its voids ratio. Porosity is the ratio of the volume of voids in a given quantity of soil to the volume occupied by the soil and its contained voids, and is expressed as a percentage. Voids ratio is the ratio of the volume of voids to the absolute, or solid, volume of soil particles in a given quantity of soil.

Supporting Power. The supporting power of a soil may be defined as its ability to resist penetration or lateral flow when loaded (see Fig. 1), and it depends upon the ability of the soil to resist shear. The shearing resistance, or shearing strength, of soils is not a single property but depends upon two other mechanical properties of soil grains, namely, *internal friction* and *cohesion.* Internal friction is the resistance of soil grains to sliding over each other; cohesion is the resistance of the soil grains to pulling apart. Internal friction depends largely upon the size, gradation, and quantity of soil particles having rough surfaces and is greatest in sand, less in silt, and least in clay. Cohesion is a measure of the stickiness of soil particles and consists of *true cohesion,* which is the molecular attraction between the soil particles, and apparent cohesion, which is caused by the molecular attraction of water. True cohesion is illustrated by two sheets of paper held together by glue; the firm surface of beach sands when wet illustrates apparent cohesion. Cohesion depends largely on the quantity of small, smooth soil particles and is greatest in clays having large proportions of colloids, less in clays having small amounts of colloids, much less in silts, and least in sands. It is obvious that internal friction in soils depends upon the density of the soil and that it varies with the amount of outside pressure exerted upon the sliding plane of soil. Cohesion is not affected greatly by outside pressure upon a sliding plane of soil. (See Fig. 2.)

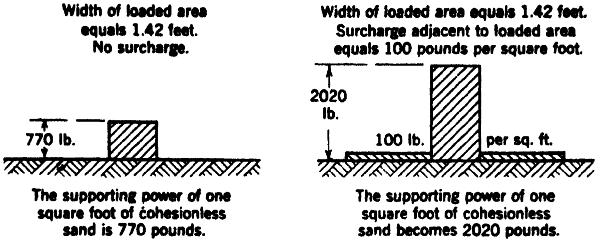


Courtesy of U. S Public Roads Administration.

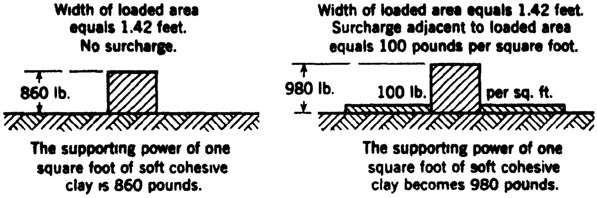
FIG. 1. Diagram illustrating the effect of loads on soils.

Permeability. Permeability is the rate at which gravitational water flows through soils; it depends upon the hydraulic gradient, upon the size and number of the soil pores, and upon the temperature. The unit of permeability is called the coefficient of permeability, is designated as K, and is the velocity in centimeters per second under a hydraulic gradient of 1. Sands and coarser soils are most permeable, and permeability decreases as the grain size decreases, clays being least permeable.

Compressibility. The compressibility of soils refers to the reduction of soil volume by the application of outside pressure. The compression of dry soils, without accompanying lateral flow, occurs when the air in the void spaces escapes or itself is compressed. Wet soils compress, without accompanying lateral flow, when the water in the void spaces can escape as pressure is applied. If sand containing water is placed in a cylinder with screens so arranged that the water may escape freely as pressure is applied through a plunger, the volume occupied by the sand will decrease rapidly, owing to the low resistance of the soil to the flow of the water through it, or owing to the high permeability of the soil. If wet clay were compressed in the same cylinder under the same conditions, the compression will occur at a much slower rate on account of the low permeability of the clay. It is thus apparent that the rate of consolidation of a soil depends on its permeability when contained moisture has an opportunity to escape.


Width of loaded area equals 1.42 feet. No surcharge.
770 lb.
The supporting power of one square foot of cohesionless sand is 770 pounds.
Width of loaded area equals 1.42 feet. Surcharge adjacent to loaded area equals 100 pounds per square foot.
2020 lb.
100 lb.
per sq. ft.
The supporting power of one square foot of cohesionless sand becomes 2020 pounds.

(*a*) Cohesionless Sand


Width of loaded area equals 1.42 feet. No surcharge.
860 lb.
The supporting power of one square foot of soft cohesive clay is 860 pounds.
Width of loaded area equals 1.42 feet. Surcharge adjacent to loaded area equals 100 pounds per square foot.
980 lb.
100 lb.
per sq. ft.
The supporting power of one square foot of soft cohesive clay becomes 980 pounds.

(*b*) Soft Cohesive Clay

Courtesy of U. S. Public Roads Administration.

FIG. 2. Relative effect on the supporting power of soils of uniformly loading the soil adjacent to the loaded area.

Elasticity. Elasticity of soils may be defined as the ability to rebound or regain part of their original form when compressed and the load is removed (see Fig. 1). No soils are completely elastic, and the degree of rebound may vary greatly. Research investigations have shown that the elasticity of soils is caused by the presence of flaky soil particles and organic colloids. The flaky particles are found frequently in silts, less frequently in clays, and rarely, if at all, in sands; organic colloids are present in peat soils.

Capillarity. Capillarity is the ability of soils to absorb water by contact with a source of supply and transmit it in all directions regardless of the amount and direction of the force of gravity. The force which causes this flow of water is surface tension, or the molecular attraction of thin films of water, and is called capillary tension. *The maximum height to which water will rise in soils is inversely proportional to the diameter of the soil pores and therefore inversely proportional to the diameter of the soil particles.* The rate at which water will rise in soils varies directly with the capillary tension (which is inversely proportional to the diameter of the soil pores) and inversely with the frictional resistance of the soil pores to the flow of water (which is directly proportional to the diameter of the soil pores or to the diameter of the soil grains); it also depends on the destruction of capillary equilibrium by surface evaporation, by freezing, or by changes in ground-water elevation. Frictional resistance to the flow of water increases more rapidly than capillary tension, and therefore there is a soil pore diameter which will produce a maximum rate of capillary rise. It has been determined both by experiment and by calculation that the maximum height of capillary rise in 24 hours occurs when the soil particles are 0.02 mm in diameter. Soil particles of this size are classified as fine silt.

TESTS OF SUBGRADE SOILS

Extensive research has shown that each of the constituents of soils influences the physical properties of soils and that the proportions of the various constituents will, to a very large extent, influence the physical behavior of soils. However, the performance of soils varies not only according to their constituents but also according to the condition or state of the soil, that is, its density, the location and quantity of ground water, the character and amount of manipulation or disturbance which will be necessary for construction, and climatic conditions. The use of the proportions of the various constituents of soils for predicting the behavior of soils is further limited by difficulties encountered in the determination of these proportions. The separation of the various grain sizes is accomplished by means of a *mechanical analysis.* The separation of grain sizes above 0.074 mm in diameter, which is the linear opening of a standard No. 200 sieve, is performed quite readily by passing the soil through sieves. Difficulties in the manufacture of sieves having extremely small openings have led to the use of sedimentation methods of determining grain sizes below 0.074 mm. Unfortunately, these methods are not entirely reliable; sedimentation rates measure spherical diameters instead of actual diameters, and are affected by the specific gravities of the particles and by the dispersion of the soil. The sizes measured are also those of the particles plus adsorbed films rather than of the particles themselves. Thus the accurate determination of grain size is limited to particles above 0.074 mm in diameter, which is approximately the dividing line between sand and silt. Since the most objectionable soil characteristics are found in the very fine-grained soils, such as silts and clays, it is obvious that present methods of separating soils into their various grain sizes are definitely limited as a means of determining soil properties. Mechanical analysis is not wholly useless, however, and is included in the routine testing of soils.

A routine of soil tests was developed as the result of extensive research, so that soil performance may now be predicted with reasonable accuracy; this accuracy will be increased as further data are accumulated. The procedure of soil testing involves the use of several terms which must be explained for the student or reader who approaches the subject for the first time. These terms are "moisture content," "subgrade soil test," "subgrade soil constant," and "soil texture."

The moisture content of a soil is the percentage of moisture in the soil, by weight, based on the dry weight of the soil. A subgrade soil test is an arbitrary test procedure for measuring a particular physical characteristic. A subgrade soil constant is either the result obtained from a subgrade soil test or a calculation based on subgrade soil tests. Texture is a term used to express the percentage of the various soil constituents and is shown graphically in Fig. 3.

The principal soil tests and constants are critical moisture, plastic limit, liquid limit, plasticity index, shrinkage limit, centrifuge moisture equivalent, field moisture equivalent, volumetric change or volumetric shrinkage, and lineal shrinkage. Density tests are also useful. It will be observed that the various subgrade soil tests are conducted on disturbed samples of soil; the practicability of testing subgrade soils in the disturbed state is obvious when attention is called to the fact that most highway subgrades are disturbed by excavation, by filling, or by some form of manipulation.

Critical Moisture. The relation between the deformation and moisture content of either confined or unconfined soils under constant load is a straight-line relationship up to a certain moisture content where the curve breaks abruptly and again follows a straight line. The moisture content at this abrupt change is known as the *critical moisture*. The critical moisture is not determined as a routine test. Tests of the U. S. Public Roads Administration show that the critical moisture is approximately equal to the plastic limit of cohesive soils and to 75 per cent of the liquid limit of cohesionless soils.

Plastic Limit. The plastic limit of a soil is the lowest moisture content at which the soil can be rolled into threads ⅛ in. in diameter without the threads breaking in pieces. The principal value of the plastic limit is that it is used in calculating the *plasticity index* of a soil. Sands, not being plastic, have no plastic limits; silts sometimes, and clays always, have plastic limits.

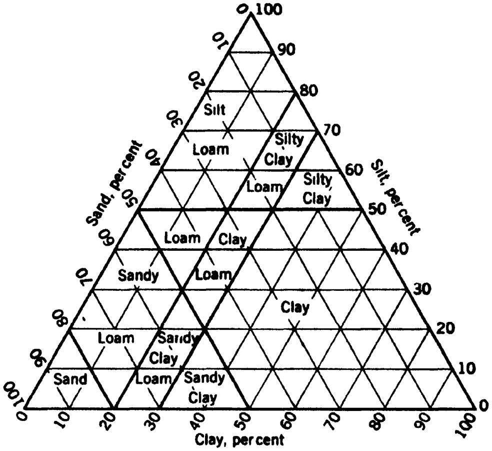


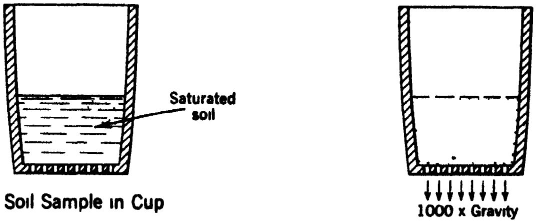
Fig. 3. Classification of soils according to grain size.

Liquid Limit. The liquid limit of a soil, as the name indicates, is the minimum moisture content which will cause the soil to flow when subjected to a very small force. The liquid limits of soils indicate the moisture contents which overcome internal friction by lubricating the grain surfaces and overcome cohesion by expanding or pushing apart the soil particles.

Plasticity Index. The plasticity index of a subgrade soil is the difference between its liquid limit and its plastic limit. Soils have little or no cohesion left when the moisture content is at the liquid limit, whereas they still have considerable cohesion when the moisture content is at the plastic limit; therefore the plasticity index offers a means of measuring the cohesion of a soil.

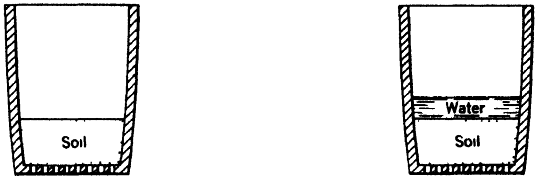
Shrinkage Limit. Soils having capillarity will shrink when deprived of their moisture by evaporation due to the capillary force exerted on the walls of the soil pores. As water is evaporated, the soil particles resist consolidation, and this resistance increases as the soil mass shrinks, until finally it equals the capillary force and further evaporation will not cause a further reduction in the volume of the soil mass. The moisture content of the soil, when these two forces become equal and shrinkage ceases, is called the shrinkage limit of the soil. Some elastic soils expand when evaporation is carried beyond the shrinkage limit; this expansion appears to be caused by mica flakes.

Centrifuge Moisture Equivalent. The centrifuge moisture equivalent of a soil is the moisture content retained by a soil which has been saturated with water and then subjected to a centrifugal force equal to 1000 times the force of gravity for 1 hour. The soil sample is placed in a cup having small openings at the bottom, as shown in Fig 4. The centrifugal force tends to squeeze the moisture from the sample at both the top and the bottom of the cup, and the flow of the moisture is resisted by the capillarity of the soil and the frictional resistance of the soil pore surfaces When the resistance to flow is large enough to overcome the centrifugal force, water is forced out at the top of the sample; this condition is known as "waterlogging." It is apparent that the centrifuge moisture equivalent is a measure of the permeability and capillarity of soils. The centrifuge moisture equivalent test is not conducted by the U. S. Public Roads Administration on soils with liquid limits greater than 35.



(*a*) Soil Sample in Cup

(*b*) Water Forced Out of Cup by Centrifugal Force.



(*c*) Sample After Centrifuging, No Water-logging

(*d*) Water-logging in Soil During Centrifuging

Courtesy of U S Public Roads Administration

FIG 4 Action of centrifugal force on sample of soil in centrifuge moisture equivalent test

Field Moisture Equivalent. The field moisture equivalent of a subgrade soil is the lowest moisture content at which a drop of water placed on a smoothed surface of the soil will not immediately be absorbed but will instead spread out over the surface and cause it to have a shiny appearance. This result indicates that the pores of sands (non-cohesive soils) are completely filled and that clays (cohesive soils) have become impervious, although their capillarity may not be satisfied.

Shrinkage. Determinations of the shrinkage of soils, to be useful for comparisons, must be based on a definite initial moisture content and must be carried to the shrinkage limit. The field moisture equivalent usually is selected as the initial moisture content. The shrinkage from this initial moisture content to the shrinkage limit is generally determined on a volumetric basis, is called volumetric change, and is expressed as a percentage of the volume of the *dried* soil. Lineal shrinkage may be calculated from the volumetric change. Sands have no volumetric change, silts undergo only small changes, and clays and colloids shrink greatly. A maximum volumetric change of about 17 per cent represents the dividing line between stable and unstable subgrade soils.

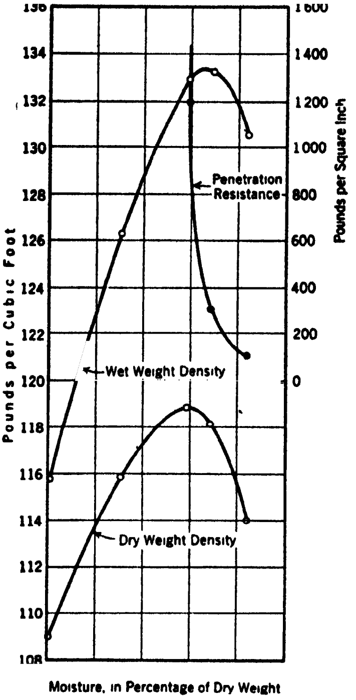
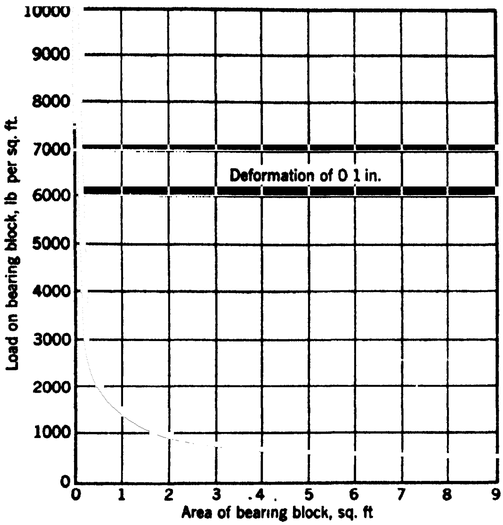


FIG. 5. Relation between moisture content and density of compacted soil for an Ohio highway embankment. (*K. B. Woods.*)

Density Tests. Tests developed by R. R. Proctor have shown that, for a given soil compacted under prescribed conditions, there is an *optimum moisture content* at which maximum density occurs. A sample of the soil at a known low moisture content is compacted by a definite procedure in a volumetric measure and its unit weight determined; the test is repeated at increased moisture contents until the unit weight decreases. The moisture-density relationship is then plotted, and the optimum moisture content, which is at the peak of the dry-weight density curve (see Fig. 5), is observed. This test is useful in determining the moisture content at which various soils should be artificially compacted.

The resistance of the soil to the penetration of a load-indicating needle may also be determined.

The degree of consolidation of the soil in an embankment which is being artificially compacted may be checked by digging a hole with a post-hole augur, weighing the soil removed, determining its moisture content, and filling the hole with dry sand of a known loose unit weight from a previously weighed quantity of the sand which is more than sufficient to fill the hole, and weighing the remaining sand.



Courtesy of U. S Public Roads Administration.

FIG. 6. Relation between the load on a given soil and the bearing area for a given vertical deformation. (*Public Roads, Vol. 6, No. 7, p. 152.*)

Direct Measurement of Subgrade Supporting Power. Numerous methods have been proposed for measuring the supporting power of subgrades by the direct application of loads to the surface of the subgrade. Circular bearing blocks or plates generally have been employed, and measurements of the vertical deformations resulting from the application of successive increments of load have been an essential feature of the procedure. The size of the bearing area exerts an important influence on the bearing value corresponding to a definite vertical deformation; this relation is illustrated by the diagram in Fig. 6, which represents the results of tests conducted by the U. S. Public Roads Administration. Housel has shown that, for any given vertical deformation of a given soil, the unit pressure is directly proportional to the perimeter-area ratio of the loaded area and that this relation may be represented graphically by a straight line. (See Fig. 7.) Thus, if a load-deformation test is conducted with two (or more) bearing blocks having different areas, the unit pressure under another area for the same deformation may be read from the diagram resulting from plotting unit pressures against the perimeter-area ratio, extending the diagram if the area at which the unit pressure is desired is larger or smaller than those employed in the tests.

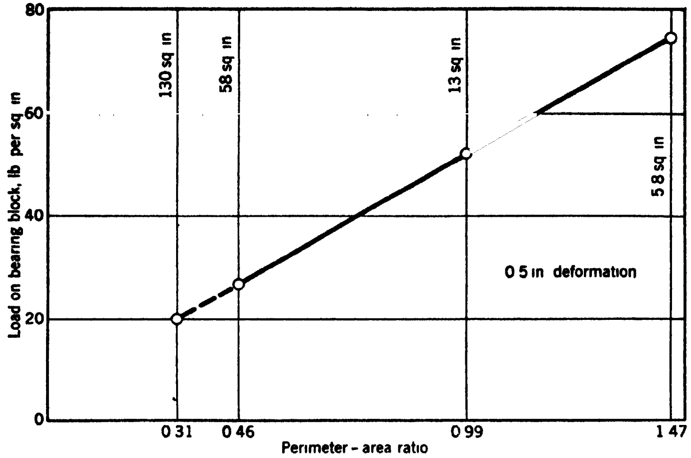


FIG. 7. Relation between the load on a given soil and the perimeter-area ratio of the loaded area, for a given deformation (*From tests by Hubbard and Field.*)

EFFECT OF VEHICLE LOADS ON PAVEMENTS AND SUBGRADES

The integrity of a highway wearing course or pavement can be preserved only when the load-carrying capacity of the subgrade is sufficient to support vehicle loads without serious deformation occurring either in the wearing surface or in the subgrade. It formerly was believed that wheel loads were transmitted through pavements or pavement foundations to the soil at angles of approximately 45° from the boundaries of the loads and that pressures on the soil were reasonably uniform The theoretical stress distribution within a homogeneous elastically isotropic mass resulting from a load applied to the horizontal surface has been determined for a point loading by Boussinesq and for a uniform load over a circular area by Love and Carothers. The Boussinesq formula, as published in the 1933 progress report of a Special Committee on Earths and Foundations of the American Society of Civil Engineers, is as follows: [1]

$$v = k\frac{W}{z^2} \tag{1}$$

k, a coefficient, is

$$k = \frac{3}{2\pi}\frac{1}{[1 + (r/z)^2]^{5/2}} \tag{2}$$

in which v = the vertical pressure, in pounds per square inch.
W = the load, in pounds.
z = the vertical distance of any given point below the surface from the vertical projection of the load, in inches.

If $R = \sqrt{r^2 + z^2}$, formula 1 becomes

$$v = \frac{3W}{2\pi}\frac{z^3}{R^5} \tag{3}$$

The Boussinesq formula is not applicable to the calculation of stresses near the surface of a loaded soil because the theory of elasticity does not apply in such locations and a so-called "concentration factor," N, has been introduced into formula 3 as follows

$$v = \frac{NW}{2\pi}\frac{z^N}{R^{N+2}} \tag{4}$$

The value of N varies from 3 to 6.

Direct measurements of actual pressures on subgrades beneath overlying pavements (that is, at the pavement-subgrade interface) have been made by various investigators utilizing pressure cells. The results of some early measurements by A. T. Goldbeck are given in Fig. 8 and Fig. 9; they show that the intensity and the lateral distribution of the pressure on the subgrade depend on the supporting power of the subgrade. Attention has already been called to the fact that the supporting power of a subgrade is due to the internal friction or cohesion of the soil (or a combination thereof) and may be increased by means of outside pressure if due to internal friction, whereas little or no increase is obtained when it is due to cohesion. This condition is illustrated in Fig. 2. The influence of internal friction and cohesion on the supporting power of subgrades may be further illustrated by considering a concentrated load (such as a loaded truck wheel) applied to the upper surface of a concrete slab (at an interior point, some distance from the edge) resting on a subgrade.

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[1] See *Soil Mechanics,* by D. P. Krynine, McGraw-Hill Book Company, New York, Chapter IV.

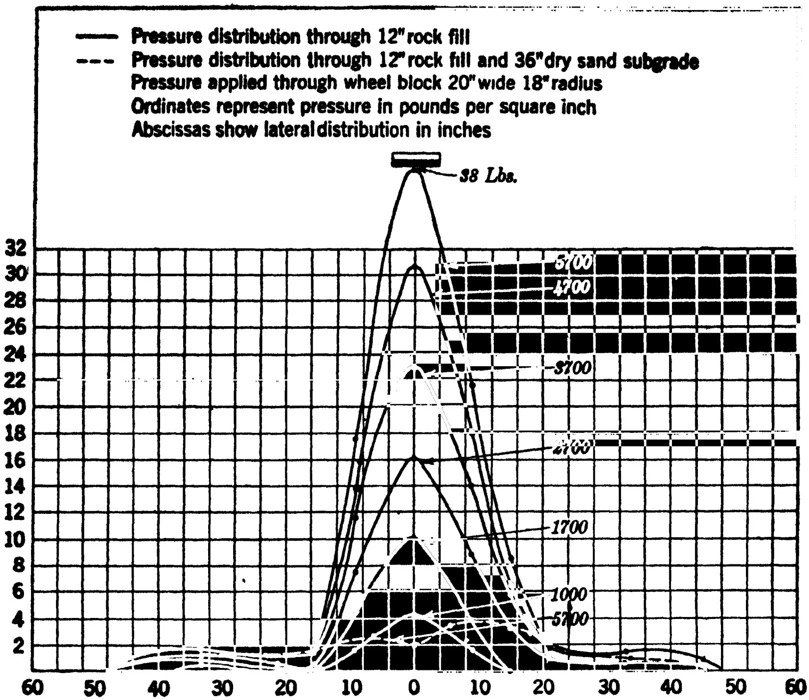


FIG. 8. Pressure distribution of various loads through a 12-in rock fill on a dry sand subgrade, the pressure being applied through a wheel block 20 in. in width and having a radius of 18 in (*Goldbeck*)

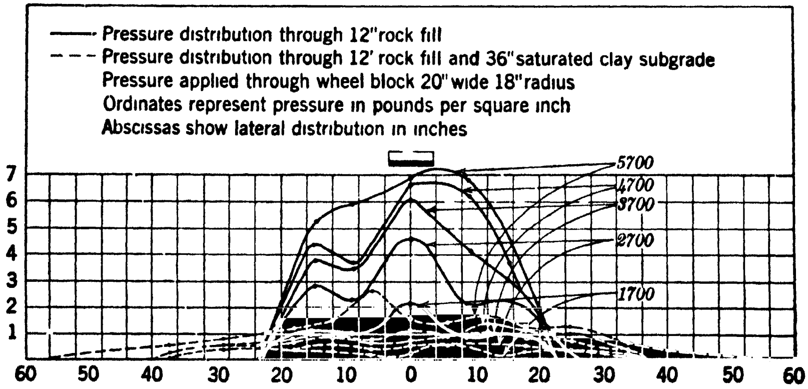


FIG. 9. Pressure distribution of various loads through a 12-in. rock fill on a saturated clay subgrade, the pressure being applied through a wheel block 20 in. in width and having a radius of 18 in. (*Goldbeck.*)

The weight of the slab acts as a distributed load (or outside pressure) surrounding the concentrated load. If the soil consists of dry sand, which has high internal friction and no cohesion, its ability to support the concentrated load is increased by the surrounding distributed load which partially confines the soil. If the soil consists of moist clay, which has cohesion but little or no internal friction, the surrounding load does not appreciably increase the supporting power of the subgrade; if the clay is moist enough to lessen the cohesion appreciably, the supporting power of the subgrade will be low, and the concrete slab will bend; if the bending strength of the slab is sufficient to support the load, the slab may remain intact for a long period of time, and the extent of the damage will consist of deformations in the subgrade unless the subgrade soil possesses considerable elasticity. That more or less permanent deformations do exist under concrete pavements on cohesive soils was demonstrated by Clifford Older on the Bates Experimental Road in Illinois. Concrete slabs do have considerable bending strength; so let us consider the case of a flexible pavement, such as a macadam (broken-stone) layer, which has low resistance to bending. If the supporting power of the subgrade depends principally on internal friction, the deflection of the macadam layer will depend upon the destruction of the internal friction by water entering the subgrade; if the subgrade soil is principally sand, the area of the macadam layer surrounding the load (assuming, of course, that the macadam layer is compact) partially confines the sand and increases the internal friction. If the soil is wet clay, no increase in supporting power is contributed by the surrounding area of the broken-stone layer; if the clay becomes wet enough to lower the sup-

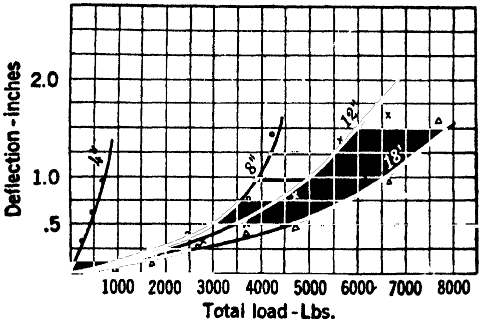
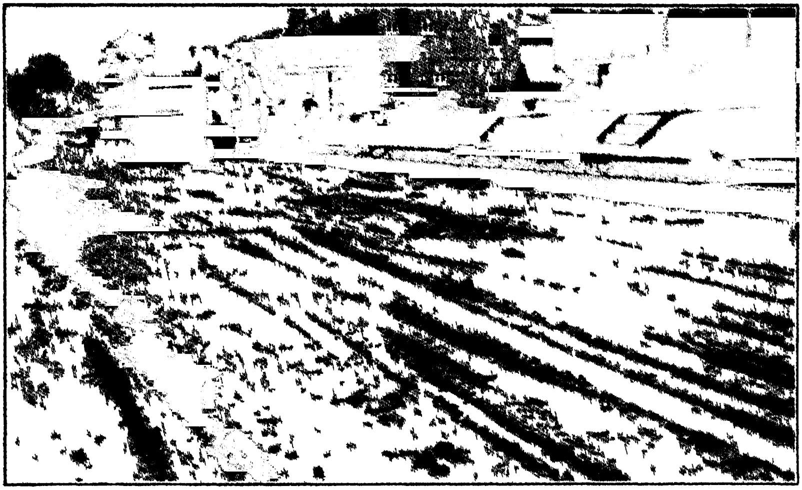


FIG. 10. Deflections of rock fills of various depths on saturated clay subgrade for various super-imposed loads. A wheel block of 20-in width and 18-in. radius was used on all depths except 4-in., on which an 8-in.-diameter circular bearing block was used. (*Goldbeck.*)

porting power very appreciably, the flexible layer bends and breaks. These conditions are illustrated clearly in Figs 8, 9, and 10. The inadequacy of increasing the thickness of flexible pavements on cohesive soils which are subject to high moisture contents is obvious, the remedy for failures under such conditions does not lie in the use of thick pavements or foundations but in some form of treatment of the subgrade

EFFECT OF THE SUBGRADE ON THE WEARING SURFACE

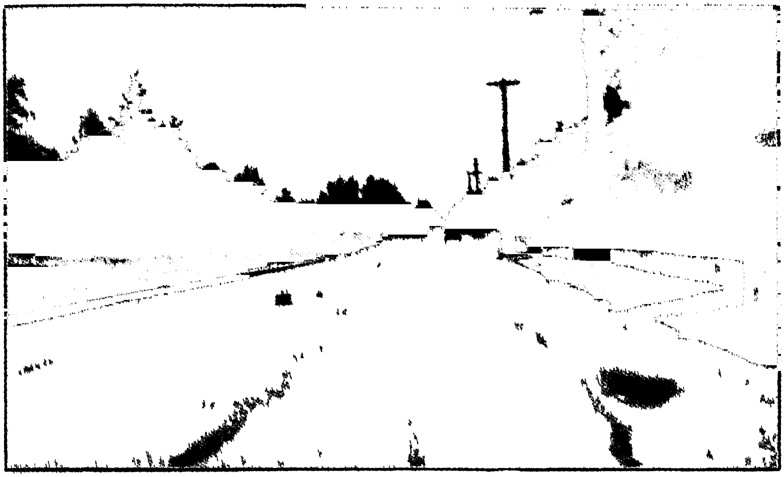
Subgrade movements which affect the integrity of highway wearing courses are (1) elastic rebound after compression, (2) shrinkage and expansion, (3) frost heave, (4) slides in cuts and fills, (5) slips, (6) settlement of fills, (7) lateral flow of soft subsoils, and (8) erosion by surface water



Courtesy of U S Public Roads Administration

Fig 11 Rebound, after rolling, of elastic subgrade

Elastic Rebound of Soils. Elastic subgrade soils may be detrimental to both flexible and rigid pavements The compaction of flexible layers on such subgrades may be difficult or impossible because the rebound after compression bends the layers upward and destroys their bond by cracking Elastic soils usually retain some consolidation after rolling, and concrete foundations or pavements may be damaged during the setting of the concrete, slight wetting of the subgrade by sprinkling or by absorption of water from freshly mixed concrete will cause rebound of the subgrade, which generally will be non-uniform, and cracking of the concrete will occur during the setting of the concrete. This cracking may extend in any or several directions. One of the most serious cases of excessive cracking of concrete pavements caused by elastic subgrades to come within the observation of the author occurred on a clay subgrade in St. Clair County, Michigan. Cracking began within 20 minutes after placing of the concrete, the cracks forming at intervals of as little as 2 to 3 in. apart and running diagonally from the center to the edges of the pavement. Measurements revealed a rebound of the moistened soil of as much as ½ to ⅝ in. within 30 minutes after moistening. Elastic subgrades may not seriously affect concrete pavements after the concrete has acquired its normal strength, but the thickness of such pavements should be greater than on non-elastic soils of high supporting ability, and some form of subgrade treatment is desirable.

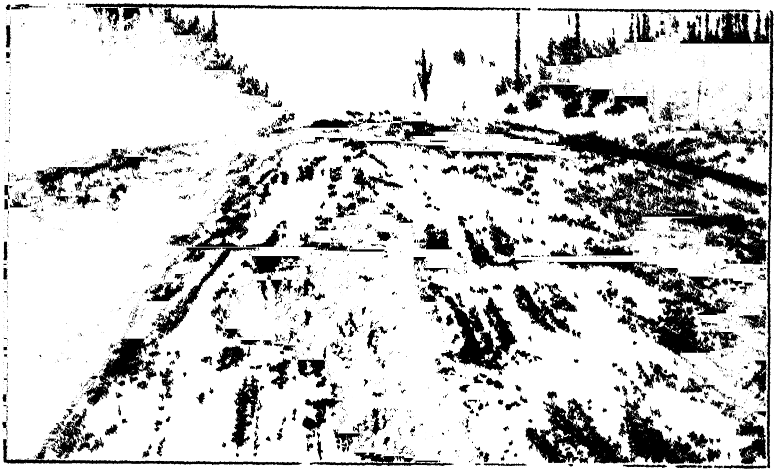


Courtesy of U. S. Public Roads Administration.

FIG 12. Cracking of concrete pavement on elastic subgrade.

Shrinkage and Expansion. The influence of capillarity on the shrinkage and expansion of soils has been described. Most sands possess little or no capillarity. Silt and friable clay soils possess capillarity, and moisture absorbed by capillary action may force the grains apart and cause sliding or caving of slopes. Subgrades composed of such soils have low supporting value when wetted by capillary action. The presence of an impervious road surface on such soils will prevent evaporation, and moisture will collect under the road surface and soften the subgrade.

Cohesive clay soils possess capillarity, but cohesion will prevent the absorption of water unless the soils are manipulated in contact with water so that mixing of the soil and water takes place. Thus, flexible pavements on cohesive clays may fail because of water reaching the subgrade through the pavement or between the pavement and the shoulders; the kneading action of vehicle loads on the pavement mixes the soil and water with resulting loss of subgrade support. The softening of natural earth surfaces on such soils by rains renders them impassable until evaporation removes the water.

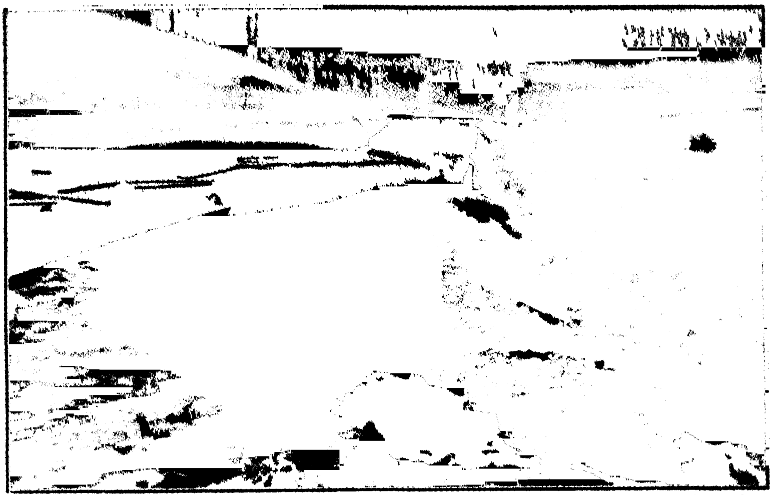


Courtesy of Michigan State Highway Department

FIG 13 Frost heave on a gravel road

Frost Heave. Damage to highway wearing surfaces and pavements by frost action is the result of heaving of the subgrade soil by frozen moisture in the form of ice layers. This heaving may occur at any time during the winter in cold climates The force exerted by the heaving soil and by the impact of traffic on affected areas shatters the wearing surface or pavement, and failure progresses when thawing occurs and when additional water from melting snows and rains enters the subgrade directly through the shattered surface and indirectly by capillary attraction from the adjacent roadsides.

Frost heave depends upon both large and small capillary openings in the subgrade soil and upon a supply of water. The water in the large capillary openings freezes at or near normal freezing temperature, whereas the water in the small capillary openings freezes at lower temperatures. As the water freezes in the large capillary openings, the resulting ice particles attract the water from nearby small openings, which then freezes, and the original ice particles increase in size. This enlargement progresses as long as capillarity continues to supply water, and eventually well-defined ice layers are built up and the soil above them heaves. Natural earth surfaces and flexible wearing courses may be greatly damaged by this heaving. Rigid pavements resist the forces of heaving but frequently are damaged to an extent which requires extensive repairs or even complete replacements.



Courtesy of U S Public Roads Administration.

FIG 14 Pavement failure caused by slide in side fill

The influence of capillarity on the occurrence of frost heave in subgrade soils is apparent from the foregoing discussion. The extent of heaving will depend upon the amount of water present in the capillary openings of the soil and upon the ability of the soil to raise additional water by capillary action. The capillary flow of moisture already has been stated to be inversely proportional to the diameter of the capillary openings and to the frictional resistance of the surfaces of the soil particles. It is evident that the greatest quantity of water for frost heave will be furnished by non-cohesive soils having high capillarity. Therefore, frost heave may be expected in silty soils which have access to ground water. Clay soils with high water table may heave in appreciable amounts, but with a low water table the amount of frost heave should be negligible.

Slides. The sliding of earth in cuts and fills occurs when the forces of cohesion and internal friction are less than the force exerted by the weight of the soil plus superimposed loads.

Slips. A slip is similar to a slide in its effects, but it results from a different condition. A slip occurs when one layer of soil moves over another layer of soil; the two layers of soil usually are different in composition and physical condition. The usual slip occurs when a layer of permeable soil moves downward over an inclined layer of impervious clay, the surface of which has been lubricated by seepage water.

Settlement of Fills. Earth swells when excavated and, when placed in fills, compacts to a degree which depends upon the nature of the soil, the moisture content of the soil, the weight of the fill, the resistance of the underlying soil, the effectiveness of artificial methods of consolidation, and the opportunity for water to escape from the fill.

Lateral Flow of Soft Subsoils. The presence of peat or muck subsoils is quite common in many parts of the United States and particularly in the glaciated areas of the northern states. Large areas may be involved, or small areas (often called "sinkholes") may be encountered. The upper layer of soil generally consists of a tough, fibrous mat, which may be several feet in thickness; below this mat the soil is black muck mixed with large percentages of water. A slimy ooze of varying thickness usually covers the firm soil at the bottom of such areas. The upper fibrous mat often possesses appreciable supporting power until it is loaded with additional earth for a highway embankment. Settlement ensues, and the soft undersoil is squeezed out gradually and frequently causes bulging of the soil surface at the sides. Stabilization of such fills by natural settlement may continue for very long periods of time, and frequent additions to the fills may be necessary.

Erosion. Erosion of non-cohesive soils occurs on side slopes, ditches, and shoulders by the action of water and, in some locations, becomes a very serious problem.

CLASSIFICATION AND IDENTIFICATION OF SUBGRADE SOILS

The objectives of soil tests are the accurate prediction of the behavior of subgrades under actual use and the determination or selection of procedures or treatments for overcoming the damaging effects of defective subgrades. The U. S. Public Roads Administration attacked the problem of predicting soil behavior by determining the soil constants of representative soil constituents, by determining the effects of various proportions of the constituents on the soil constants, and by testing large numbers of soils whose performance was known. The influence of the various constituents on the physical properties of soils has been indicated, but it will be desirable to summarize these influences as follows:

1. Gravel and sand contribute high internal friction.
2. Silt, peat, and diatomaceous particles contribute detrimental capillarity.
3. Clay and colloids contribute cohesion, and, mixed with silt and not flocculated, contribute compressibility.
4. Mica flakes and other flaky or scale-like particles, and organic matter, contribute elasticity.

U. S. Public Roads Administration Classification. The results of these soil studies by the U. S. Public Roads Administration led to the classification of subgrade soils into eight uniform and three nonuniform groups, based on soil constituents, soil characteristics, and subgrade performance. This classification is given in Table 1. The mechanical analyses, or grading, and the test constants of the uniform groups are summarized in Table 2. The grading of good soil mortars in group A-1 subgrade soils is shown graphically in Fig. 16.

The values of the test constants are useful in the identification of subgrade soils. The U. S. Public Roads Administration found that the interrelationship of various soil constants was of equal or even greater importance than the individual constants themselves in the identification of subgrade soils. The four diagrams of Fig. 15 show the relationships which were found experimentally to exist between the liquid limit and four other soil constants for the eight uniform subgrade groups described in Table 1. These four diagrams constitute the soil-identification chart of the U. S. Public Roads Administration. The classification of subgrade soils given in Table 1 was not intended to be a rigid and final classification and has been modified by some highway organizations; for example, some state highway departments consider it desirable to divide Group A-4 subgrades into two separate divisions.

PREPARATION AND STABILIZATION OF SUBGRADES

The highway engineer usually is confronted with two types of problems in connection with subgrades: (1) the correction of defects in subgrades already in use, and (2) the prevention of unsatisfactory conditions on new improvements. The first type of problem manifests itself by damage to the pavement, wearing course, wearing surface, or some other part of the highway structure, as the result of shear failure, abrasive failure, elastic action, shrinkage, slides, or incomplete consolidation of the subgrade; the causes of the condition must be

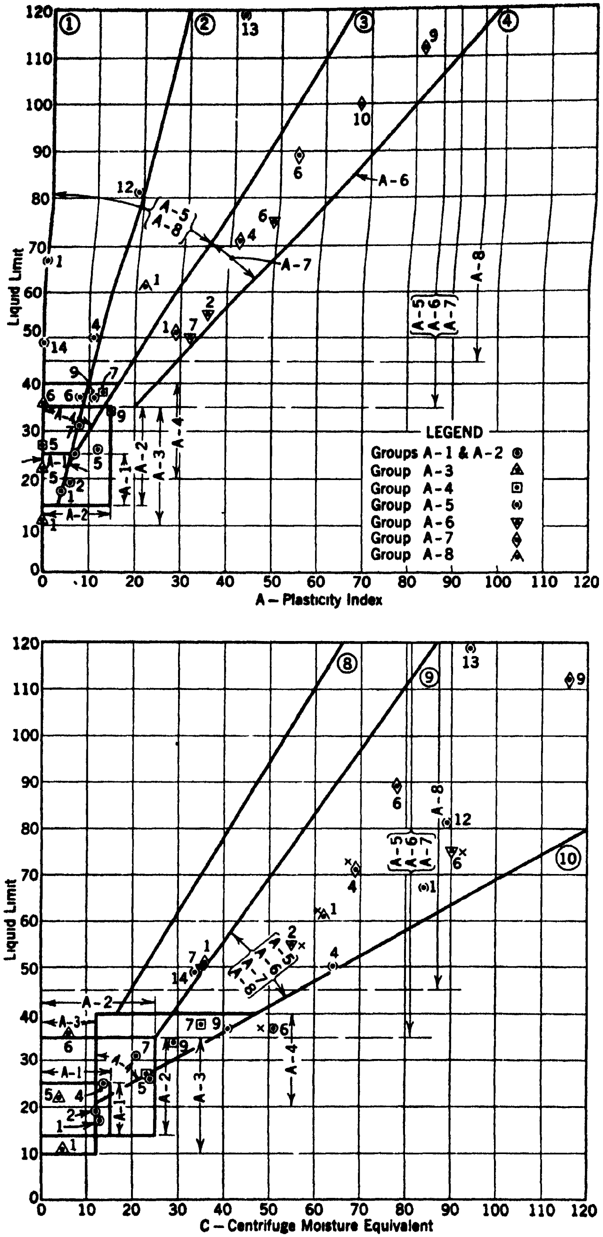


Fig. 15. The subgrade soil identification

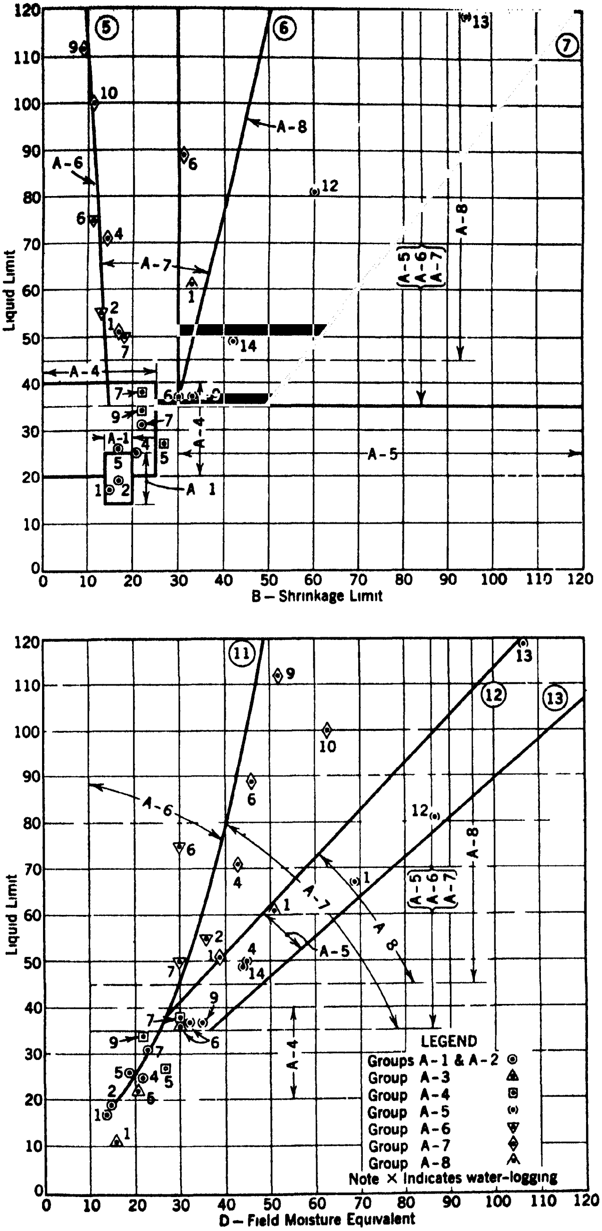


chart of the U S Public Roads Administration

TABLE 1

Classification of Subgrade Soils by U. S Public Roads Administration

Group	Soil Constituents	Soil Characteristics	Subgrade Performance
		Uniform Subgrades	
A-1	Well-graded materials, sand, silt, and clay, excellent binder	High internal friction, high cohesion, no detrimental shrinkage, expansion, capillarity, or elasticity	Highly stable under wheel loads, irrespective of moisture conditions Functions satisfactorily when surface-treated or when used as a base for relatively thin wearing courses
A-2	Coarse and fine materials, improper grading or inferior binder	High internal friction and high cohesion only under certain conditions May have detrimental shrinkage, expansion, capillarity, or elasticity	Highly stable when fairly dry Likely to soften at high water content caused either by rains or by capillary rise from saturated lower strata when an impervious cover prevents evaporation from the top layer, or to become loose and dusty in long-continued dry weather
A-3	Sand, no binder	High internal friction, no detrimental capillarity or elasticity	Lacks stability under wheel loads but is unaffected by moisture conditions Not likely to heave because of frost nor to shrink or expand in appreciable amount Furnishes excellent support for flexible pavements at moderate thickness and for relatively thin rigid pavements
A-4	Cohesionless silts, friable clays No appreciable amount of sticky colloidal clay	Internal friction variable, no appreciable cohesion, no elasticity, capillarity important	Has a tendency to absorb water very readily in quantities sufficient to cause rapid loss of stability even when not manipulated When dry or damp, presents a firm riding surface which rebounds but very little upon the removal of load Likely to cause cracking in rigid pavements as a result of frost heaving, and failure in flexible pavements because of low supporting value
A-5	Micaceous and diatomaceous silts and sands	Similar to A-4 and in addition possesses elasticity in appreciable amount	Similar to Group A-4 but furnishes highly elastic supporting surfaces with appreciable rebound upon removal of load even when dry Elastic properties interfere with proper compaction of macadams during construction and with retention of good bond afterwards
A-6	Cohesive clays, dispersed state	Low internal friction, cohesion high under low moisture content, no elasticity, likely to expand and shrink in detrimental amount	In stiff or soft plastic state absorb additional water only if manipulated May then change to liquid state and work up into the interstices of macadams or cause failure due to sliding in high fills Furnish firm support essential in properly compacting macadams only at stiff consistency Deformations occur slowly, and removal of load causes very little rebound Shrinkage properties combined with alternate wetting and drying under field conditions are likely to cause cracking of rigid pavements
A-7	Micaceous, diatomaceous, and flocculated clays May contain lime or associated chemicals productive of flocculation in soils	Similar to A 6 but possess elasticity also	Similar to Group A-6, but at certain moisture contents deforms quickly under load and rebounds appreciably upon removal of load, as do subgrades of Group A-5 Alternate wetting and drying under field conditions lead to even more detrimental volume changes than in Group A-6 subgrades May cause concrete pavements to crack before setting and to crack and fault afterwards
A-8	Peats and mucks	Low internal friction, low cohesion, apt to possess capillarity and elasticity in detrimental amounts	Very soft incapable of supporting a road surface without being previously compacted
		Non-Uniform Subgrades	
B-1	Non-uniform natural ground due to abrupt variation in soil characteristics or soil profile, or to frequent change in field conditions		Soils of these groups cause concrete pavements to crack or fault excessively and flexible types to fail or to develop rough riding surfaces
B-2	Non-uniform subgrade due to non-uniform composition of fill		
B-3	Non-uniform subgrade consisting in part of natural ground and in part of fill materials		

ascertained by a careful survey and such soil tests as may be considered necessary. The kind and extent of treatment necessary in the second type of problem are determined by means of a careful and thorough survey of the proposed subgrade and by tests of carefully selected samples of the subgrade soils; the construction of a soil profile may be a desirable feature of the soil survey. For a description of procedures employed in soil surveys, the reader is referred to the current Standards of the American Association of State Highway Officials and the American Society for Testing Materials.

The beginner will avoid some confusion if he will consider methods of preparing and stabilizing subgrades under the classifications: (1) those which are designed primarily to prevent damage from above, and (2) those which are designed primarily to prevent or correct damage from conditions which exist below the surface. Some methods of preparation and stabilization are designed to enable the ground surface to be used as the travel surface, whereas other methods are designed to provide the subgrade with sufficient resistance to loads and subgrade movements to support the superimposed wearing course or pavement. The two procedures may, of course, be combined on one road.

Methods of preparation and stabilization of subgrades include:

1. Drainage.
2. Removal and replacement of defective soils.
3. Mechanical manipulation.
4. The use of blanket courses.
5. The use of foundation courses.
6. Surface treatment.
7. Subsurface waterproofing.
8. The use of admixtures.
9. Consolidation.
10. Measures for the prevention and correction of slides and slips.
11. The control of erosion.

Drainage. Drainage, one of the most effective methods of stabilizing subgrades, is accomplished by means of open ditches, blind drains, and tile or pipe drains. Drainage always is effective in the removal of surface water, but the successful removal of ground water depends upon the capillarity of the subgrade soil. The removal of ground water from Group A-1 and Group A-3 subgrades may be entirely practicable because of the absence of capillarity. The successful application of drainage methods to Group A-2, Group A-4, and Group A-5 subgrades depends upon the lowering of the water table to depths greater than the effective capillary rise. The subsurface drainage

TABLE

TYPICAL GRADINGS AND SOIL CONSTANTS

U. S. Public

Group	Typical Grading	Liquid Limit	Plasticity Index
A-1	Not over 50% retained on No. 10 sieve Composition of soil mortar (material below No. 10 sieve): Clay 5 to 10% Silt 10 to 20% Total sand 70 to 85% Coarse sand (above No. 60 sieve) 45 to 60%	Generally 14 to 25	Usually less than 8; see curve 2, Fig. 15
A-2	Not less than 50% sand in the material below No. 10 sieve	Generally 14 to 35	Usually less than 15
A-3	Effective size generally not less than 0.10 mm	About 10 to 35. Low limits indicate beach and other rounded sand grains; high limits indicate high degree of surface roughness or angularity	Have none
A-4		Generally 20 to 40	Less than values of curve 3, Fig. 15
A-5	Usually less than 55% sand	Usually greater than 35	Seldom greater than curve 3
A-6	Usually over 30% clay	Usually greater than 35	Approximately equal to curve 4
A-7		Usually greater than 35	Generally greater than curve 3 and smaller than curve 4
A-8	Not significant	Likely to be greater than 45	Generally less than curve 3

2

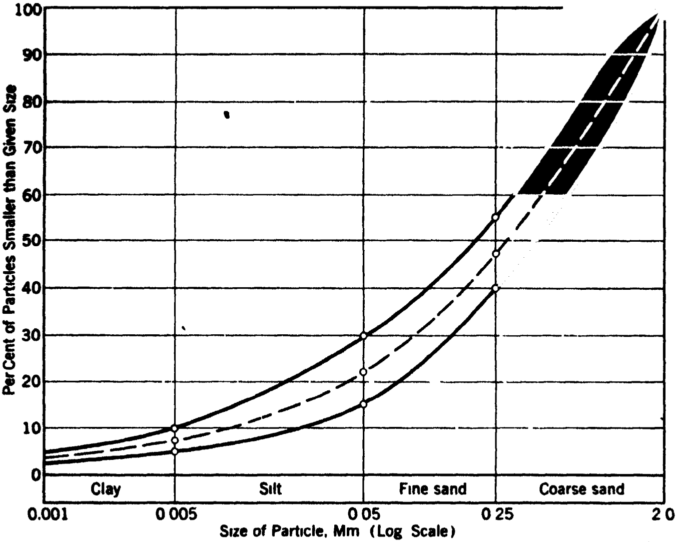
OF UNIFORM SUBGRADE SOIL GROUPS

Roads Administration

Shrinkage Limit	Centrifuge Moisture Equivalent	Field Moisture Equivalent
Generally 14 to 20	Seldom more than 15	Not significant
May be either theoretical or significant, depending upon other constants Unlikely to exceed 25 when significant	Unlikely to exceed 25	Not significant
No significant shrinkage limit	Unlikely to exceed 12 In combination with the liquid limit, discloses the relative resistance to flowing possessed by sands equal in degree of capillarity (see Fig 15)	Indicates porosity when completely saturated, in combination with liquid limit discloses degree of saturation required to cause sands to have very small shear resistance
Generally less than 25 Increase in expansive properties of members of this group indicated when shrinkage limits exceed 20 and approach relationship shown by curve 6, Fig 15	Generally greater than 12, approaching values indicated by curve 10, Fig 15, but unlikely to waterlog, although exceptions occur When greater than the liquid limit in absence of waterlogging, indicates especially unstable silts	When approximately equal to or larger than centrifuge moisture equivalents, indicates presence of expansive properties in detrimental amounts
Generally greater than 30 and greater than 50 for very undesirable members of this group May approach relation indicated by curve 6, Fig 15, for silts containing peat and approach relation indicated by curve 7 for soils containing either diatoms or mica in appreciable amount	Greater than 12 and unlikely to waterlog, although exceptions occur Often has values between curves 9 and 10, Fig 13, for clay soils containing sand in appreciable amount Likely to waterlog when exceeding 40	May approach values indicated by curve 12, Fig 15, for silts containing peat in appreciable amount and those indicated by curve 13 for highly elastic silts containing mica or diatoms in appreciable amount May not exceed those indicated by curve 11 for kaolins having good binder properties
Unlikely to be much in excess of values of curve 5	May approach values indicated by curve 10 for the highly colloidal soils, with values lying between curves 9 and 10 for clay soils containing sand in appreciable amount Likely to waterlog when exceeding 40	May approach values indicated by curve 11, generally, but smaller when the grading of the colloidal clay soils of this group is such as to cause smoothed surface of soil when wetted to become highly impermeable
Flocculated soils may slightly exceed values of curve 5 soils containing mica, peat, or diatoms may greatly exceed values of curve 5, but generally less than curve 6, soils subject to frost heave have the higher values	Generally between curves 9 and 10	Soils either flocculated or having organic matter partially decomposed into the colloidal state may have, like Group A–6 soils, shrinkage limits approaching curve 5 Their field moisture equivalents, however, are likely to be appreciably greater than those of curve 11
Generally near, but seldom much greater than, curve 6	Generally between curves 9 and 10	Same as Group A–5

of Group A-6 and Group A-7 subgrades is not possible, but surface methods of drainage such as ditches and gutters, tile drains at the junction of shoulders and wearing courses to prevent water collecting between the pavement and the subgrade, and oiling of the shoulders are effective. Drainage methods and their applications will be discussed in Chapter III.

Removal and Replacement of Defective Soils. The removal and replacement of unsatisfactory soils is expensive when large volumes of material are involved and usually is confined to silt pockets or small muck or peat deposits. Sometimes large deposits of muck or peat in swamps or "sinkholes" are excavated and replaced with good material; in other cases, explosives are used to displace the muck which is then replaced with a fill of satisfactory soil.



Courtesy of U. S. Public Roads Administration.

FIG. 16. Grading of good soil mortars, representative of material below No. 10 sieve in Group A-1 subgrade soils.

Mechanical Manipulation. The mechanical manipulation of subgrades consists of scarifying, harrowing, and rolling and is usually performed in order to increase the uniformity of the upper portion of the subgrade and sometimes to increase the density of the soil. These operations usually precede the placing of wearing courses on subgrades whose surface condition is not uniform.

Blanket Courses. The term "blanket courses" usually refers to rather thin layers of material placed on subgrades to distinguish them from thick layers or foundations. Blanket and foundation courses should not be confused with replacement of inferior soils.

Blanket courses frequently are necessary or desirable on Group A-3 subgrades prior to the construction of macadam wearing courses to prevent the soil from working up into the interstices of the macadam layer and preventing consolidation. Several materials may be used, depending upon their availability; they include clay, stone screenings, straw, marsh hay, grape vines, bank or pit gravel containing considerable clay, and topsoils.

Blanket courses on Group A-6 and Group A-7 subgrades should consist of fine granular material which will act as an obstruction to prevent the soil from working up into the interstices of macadam wearing surfaces. Oiling of such subgrades prior to placing blanket courses also is beneficial.

Foundation Courses. Foundation courses are distinguished from blanket courses by their thickness, which may vary from 3 or 4 in. up to 2 ft or more. They may consist of porous, granular material (such as sand, gravel, cinders, or broken stone or slag), of dense mixtures of granular material and cohesive material (such as sand-clay and sand-clay-gravel mixtures), of Portland cement concrete, of asphaltic concrete, or of bituminous macadam. Foundation courses are applicable to subgrades belonging to Groups A-4, A-5, A-6, and A-7. The placing of thick foundation courses of granular material (gravel or broken stone) on subgrades which have undergone detrimental frost heave has produced very satisfactory results. Thicknesses of 1 to 3 ft are advisable, and drainage of the foundation courses is an essential detail of construction.

Surface Treatments. The surface treatment of subgrades consists of the application of a thin layer, usually of bituminous material, to the surface of the subgrade or of the shoulders in order to waterproof the surface and prevent softening by rains. Oiling is the most common method of surface treatment and has been used on earth roads as well as on subgrades below wearing courses. The extensive oiling of natural earth surfaces has been practiced in some locations for many years to prevent softening in wet weather and the raising of dust in dry weather.

The oiling of subgrades prior to the placing of wearing courses has not been practiced to as great an extent as the oiling of earth roads, but it is increasing. The oiling of clay subgrades before placing gravel wearing courses has been found effective in reducing the tendency of the soil to work up into the gravel layer during wet weather. Tar-paper coverings have been used on Group A-7 subgrades on which concrete pavements or foundations were being constructed for the purpose of preventing moistening of the elastic soil by water from the fresh concrete; this practice, however, has not always been completely effective.

Subsurface Waterproofing. The subsurface waterproofing of subgrades consists of the application of a layer of bituminous material at some distance (usually over 12 in. and up to 3 or 4 ft) below the surface of the subgrade. The application may be made by loosening the soil and, by means of a heavy grader, transferring it from one side of the subgrade to the other and applying the bituminous material to half the width at a time. These treatments have been used on subgrades composed of soils having sufficient capillarity to cause appreciable frost heave in cold climates. The results in many cases have been disappointing when the original soil was used above the bituminous layer, probably because rainwater from above and ground water from the sides have softened the replaced soil. The utilization of this method in connection with blanket and foundation courses or in conjunction with the application of bituminous admixtures to the original soils, although somewhat costly, seems to possess merit and may warrant thoughtful consideration on many locations, particularly in the early steps of stage construction.

Use of Admixtures. An ideal subgrade soil consists of a mixture of sand, silt, and clay, in which the internal friction provided by the sand and the silt, supplemented by the cohesion of the clay, furnishes supporting power, and the expansion of the clay, when wetted, is just sufficient to prevent water from entering and softening the soil. Natural soil mixtures having the desired grading and binder properties are lacking in many locations, and admixtures of various materials may be used to improve the stability of these defective soils.

The successful use of admixtures depends upon the absorptive properties of the soils with which they are mixed. Some soils have a strong attraction for air; sometimes this attraction is so strong that drops of rain will roll along on dry dust without wetting it. Similarly, other soils have a strong attraction for moisture, as, for example, dried clods which retain enough moisture to enable them to resist crushing to a very great extent. Other soils have a strong affinity for bituminous materials. Also, soils vary in their ability to exchange one adsorbed material for another. Admixtures may be classified as follows:

1. Soil materials to provide the characteristics in which a given subgrade may be deficient.
2. Deliquescent materials, such as calcium chloride and com-

mon salt, which provide moisture films of great adhesive strength.
3. Materials which facilitate the replacement of adsorbed air, or other films with moisture or other materials having high adhesive strength. These materials include soaps, limes, pure limestone dust, and hydrated lime.
4. Adhesive materials such as Portland cement and bituminous materials which bind the soil grains together and prevent the absorption of water.

The relative affinity of soils for water and bitumen may be determined by mixing a quantity of the soil with the bitumen proposed for use as an admixture, agitating the mixture in water at 140° F for 5 minutes, and determining the percentage of soil particles which separate out after the bituminous film is displaced by the water. This test, known as a bitumen-water preferential test, has been used most extensively in connection with asphalt products.

Another test for determining the value of bituminous materials for admixtures is known as the swell test and is conducted by compressing a mixture of the soil and the bitumen in a cylindrical mold and measuring the vertical swelling which occurs when the specimen (held in the mold) is placed in contact with water at both its top and bottom. The maximum swell of a 4-in.-diameter specimen weighing 1000 grams should not exceed about 0.01 in.

Admixtures are used both for surface stabilization to provide relatively low-cost roads for light to moderate traffic and for stabilization of subgrades which will support various types of wearing courses. The use of admixtures will be further described in Chapter VIII.

Consolidation of Subgrades. The consolidation of subgrades ordinarily is accomplished by scarifying, harrowing, and rolling, or by the action of traffic supplemented by occasional smoothing with blade graders or scrapers.

The consolidation of fills of appreciable thickness formerly was left to nature, even on heavily traveled highways, and the construction of pavements on such fills often was deferred (sometimes for periods as long as three to four years) until a stable condition was indicated by a series of levels taken over a period of time. On many lightly traveled land service roads this procedure now is necessary for reasons of cost. The artificial consolidation of fills on moderately or heavily traveled highways is desirable and now is commonly practiced. The procedure will be described in Chapter V.

Prevention and Correction of Slides and Slips. The failure of a slope in a cohesionless soil by sliding may be avoided by constructing a cut or an embankment so that the angle of the side slope with a horizontal plane is greater than the angle of repose of the soil, and by protecting the slopes from erosion and the penetration of water. A slope in a cohesionless soil may be of any height provided that the angle of repose is not exceeded. The failure of a slope (in either a cut or an embankment) in a cohesive soil occurs when the shearing strength of the soil resulting from both internal friction and cohesion is overcome. The slope of a cohesive soil is limited in height. Slides in cuts are prevented by limiting not only the angle but also the height of slopes. Terracing sometimes is practiced to control the height of slopes, and erosion control always is important. Slides in embankments are prevented by several means including the construction of the embankments with satisfactory soil, artificial compaction, designing the slopes to withstand the shearing forces involved, and adopting appropriate measures for the control of erosion.

The existence of conditions favorable to slips usually is manifested by movement of soil layers but may be determined by means of borings over the suspected area. Treatments for the prevention of slips include (1) drainage of the slip plane by intercepting the water before it reaches the slip plane, (2) roughening the slip plane by blasting, (3) driving piles at the toe of slopes, and (4) constructing retaining walls at the toe of slopes. Drainage of the slip plane sometimes is difficult, especially if the upper soil layer is porous up to the ground surface and embraces a large area, and several intercepting drains may be needed. Piling is not always effective but has been successful in some locations. Retaining walls are usually effective if the height and length of the material held is not too great; drainage of retaining walls is essential.

Control of Erosion. Various devices employed to reduce the erosion of soils include intercepting ditches, the paving of ditches, gutters, and spillways, the selection of suitable materials for fills, the use of pipe drains in place of ditches, the oiling of shoulders, the placing of riprap on banks, and the seeding and sodding of slopes; they will be further described in Chapter IV.

Miscellaneous Methods. Various other methods for stabilizing subgrades include heating of the subgrade soil and electrochemical treatment. Their application thus far has been very limited.

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CHAPTER III

DRAINAGE

Water reaches roadways principally by the precipitation of rain and snow on the road area and by the flow of surface waters from adjacent lands, but also by the overflowing of watercourses and by percolation through the soil. The removal of surface water is known as surface drainage and is accomplished by means of crowned road and street surfaces, shoulder and sidewalk slopes, side slopes, ditches, gutters, and pipe drains. The removal of water percolating through the soil (i.e., *ground water*) is known as underdrainage and is accomplished by means of ditches, blind drains, and tile drains. Both surface water and ground water must be carried to outlets, which may be either natural or artificial watercourses or lakes. When watercourses intersect the roadway, bridges or culverts must be used to carry them beneath the travel surface; watercourses and the bridges and culverts across them are sometimes described as cross-drainage.

DITCHES AND GUTTERS

No standard distinction between ditches and gutters has been adopted, and most people consider a gutter to be a shallow ditch; sometimes paved ditches are called gutters.

Side Ditches for Surface Drainage. Side ditches are used along the roadway in cuts and on ground that is comparatively level, and sometimes adjacent to embankments to prevent roadwater from flowing over cultivated or otherwise improved land. Ditches usually are V-shaped or trapezoidal in cross-section, the former being generally preferred because they can be both constructed and maintained by means of blade graders. The sides of ditches should have flat slopes because of their safety and also because they may be more easily and economically mowed by either team or power mowers. Minimum slopes of 4 horizontal to 1 vertical are desirable for through highways, and many primary state highways are constructed with much flatter slopes. The use of wide, deep side ditches for carrying large volumes of surface water is not good practice on account of the hazard to traffic and possible damage to the roadway; when the water cannot be diverted into cross-drainage channels, it is desirable to obtain additional right-of-way and place the ditch as far as possible from the roadway. This procedure often is impracticable on secondary county roads and land-service roads, and the only safe alternative consists of installing substantial guard rails and using as flat a slope as possible on the adjacent roadway and shoulders.

Side Ditches for Underdrainage. Deep side ditches have been used for the underdrainage of subgrades but are not good practice for the reasons previously stated, in addition, particularly if the excavated earth is not needed for embankment, they frequently are more costly than tile drains.

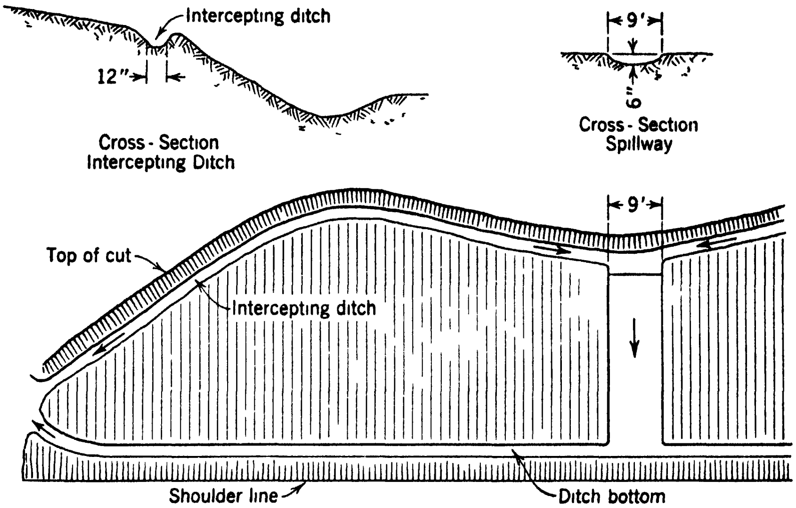


FIG 17 Intercepting ditch and sodded spillway on outside slope

Intercepting Ditches. Intercepting ditches are used on natural slopes to prevent erosion of the artificial slopes in cuts and to relieve side ditches and pipe drains. Trapezoidal cross-sections are used, and the excavation usually must be done by hand labor.

Paved Ditches and Gutters. Paved ditches and gutters are used to prevent erosion of the soil. Concrete, and cobblestones set in mortar, are most widely used. A 5- or 6-in. thickness of concrete is usually sufficient; expansion and contraction joints are desirable, and reinforcement dowels at expansion joints are frequently used.

Ditch Checks. For economic reasons, ditch checks often are installed for preventing scour in ditches instead of using pipe drains or paved ditches. Concrete and creosoted lumber are most commonly employed for this purpose A typical creosoted lumber ditch check

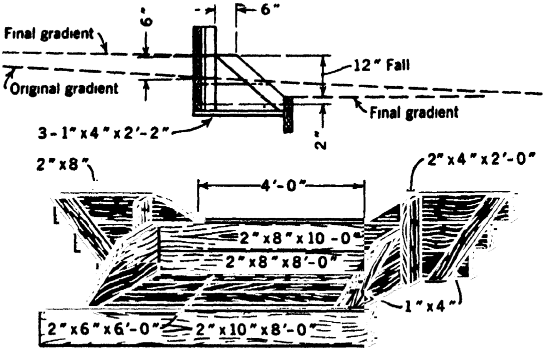
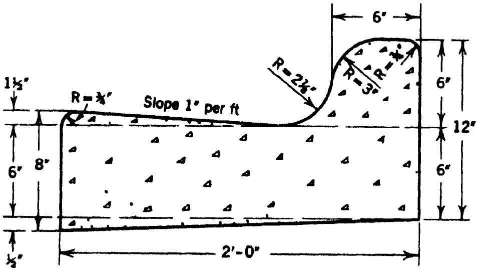
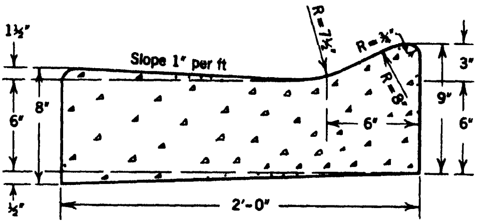


Fig 18 Creosoted lumber ditch check for reducing erosion (*Sedgewick County, Kansas, design, from Wood Preserving News*)

is shown in Fig 18 A series of checks is constructed so that the grade is held flat enough between checks to prevent serious erosion,



(*a*) Full Section



(*b*) Driveway Section

Courtesy of Louisiana Highway Commission

Fig 19 Combination concrete curb and gutter

generally grades are held to 1 per cent or less. It is desirable to restrict the fall to a maximum of 12 in.

Curbs and Gutters. The combination concrete curb and gutter, illustrated in Fig. 19, is widely used on urban streets to carry off surface water and to provide a pleasing border between the traveled surface and the sidewalk or side area, but it is not suitable for use on rural highways because it may cause accidents, particularly at night. The lip curb, constructed monolithically with concrete pavements, is satisfactory for rural highways; it is illustrated in Fig. 20. A shallow V-shaped concrete gutter also is used adjacent to concrete as well as other types of pavements. These types of gutters do not create traffic hazards and are widely used in both cuts and fills to prevent erosion; they also reduce the amount of excavation in cuts in comparison with ordinary sections. There is some difference of opinion in regard to the necessity of widening pavements constructed with monolithic lip curbs; the actual reduction in the traveled width is not great, and, when the traffic lanes are 10 ft in width, the necessity of widening may be questionable; when the width of the traffic lanes is 9 ft, however, widening equivalent to the width of the lip curb would appear to be desirable. The driveway section of the combination concrete curb and gutter, shown in Fig. 19(*b*), could be used as a separate curb and gutter for rural highways; this section can be modified for widths of 9 in. to 24 in. The use of gutters adjacent to the pavement on rural highways requires spillways or inlets for removing the water quickly. The use of paved spillways at intervals along embankments creates hazards to vehicles which run over the curbs onto the shoulders; warning signs are usually placed on such locations.

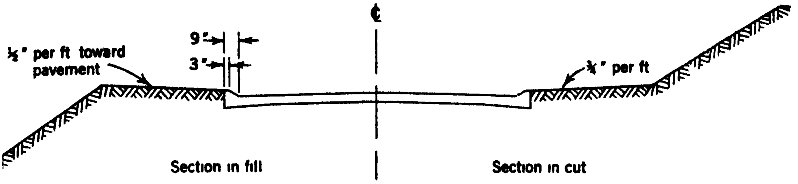


FIG 20 Integral lip curve on concrete pavement.

PIPE DRAINS

Pipe Drains. Pipe drains, or storm sewers, are widely used for removing surface water and are desirable for carrying the water along roadways when sufficient widths of right-of-way are not available for suitable ditch construction. Vitrified clay tile and sewer pipe and concrete sewer pipe are most desirable; occasionally soft-burned clay tile (farm tile) are used. Sewer pipe is preferable to tile with fully straight sides because the joints can be calked to prevent the entrance of silt and roots; tile is used because of its lower cost and in firm soils may be entirely adequate; however, the upper half to two-thirds of the joints should be covered with tar paper to prevent silt from entering. Soft-burned clay tile should never be used above the frost line.

Corrugated metal pipe sometimes is used for pipe drains in soils where settlement may occur. All sections of corrugated metal pipe should be connected with metal bands.

Inlets, Catch Basins, and Manholes. Inlets are the openings from gutters or ditches into pipe drains or culverts and are of various types. The inlet from a ditch into a pipe drain along a rural highway may consist merely of a straight or V-shaped concrete headwall with or without a paved approach. Drop inlets are used where the water from a gutter or ditch must be discharged into a drain or a culvert at a lower elevation.

Catch basins or manholes usually are circular in shape and generally are connected with drop inlets of pipe drains. Long pipe drains should have manholes at intervals of 500 to 600 ft to facilitate inspection and cleaning. Catch basins or manholes with "drop bottoms" to serve as collectors for sand and silt are sometimes recommended for pipe drains connected with rural highways; the necessity for this type of catch basin, in the author's experience, is infrequent, and, if possible, attempts should be made to place pipe drains with sufficient fall to make them largely self-cleaning. An exceedingly large maintenance force would be necessary after rains if much sediment collected in catch basins unless the drop bottoms were made extremely deep. There are, of course, locations where the drop-bottom type of catch basin is warranted on account of grades and soil conditions, but paving of gutters and ditches may avoid the necessity of frequent cleaning.

Blind Drains for Underdrainage. Blind or French drains consist of trenches filled with coarse gravel or broken stone. Large pieces of broken stone or gravel (3 to 6 in. in diameter) are placed in the bottom of the trench and are covered with smaller pieces (above ¾-in. diameter). Blind drains are not as effective as tile drains but may be less costly when suitable materials are available; they are effective for small quantities of water.

Tile Drains for Underdrainage. Soft-burned clay tile (farm tile) formerly was used almost exclusively for underdrainage because it was believed that a large amount of water entered the drains through the porous walls of the pipe. Most of the water enters pipe drains through the joints between the sections of pipe, and the only reason for using unglazed clay tile is its cost. Some unglazed or medium-burned tile that is almost as good as vitrified tile is obtainable in some locations. The softer grades of unglazed farm tile should not be used if they are to be placed above the normal frost line. Concrete tile is widely used and is satisfactory if the concrete is of good quality. Weak, porous concrete disintegrates when subjected to frost action. Perforated metal pipe also is used for underdrains.

The pipe for pipe underdrains should be laid carefully after a layer of gravel or crushed stone has been placed in the trench to a depth of 1½ to 2 in. The trench then should be backfilled with coarse stone or gravel (¾ to 3 in. in diameter) to within 6 or 8 in. from the top. The top of the trench usually is covered with part of the material excavated from the trench. When gravel and stone are costly, coarse sand sometimes is substituted but is not as effective. When sand is used for filling or when there is danger of fine-grained soil entering the pipe from the sides of the trench, the upper two-thirds of the pipe joints should be covered with strips of tar paper.

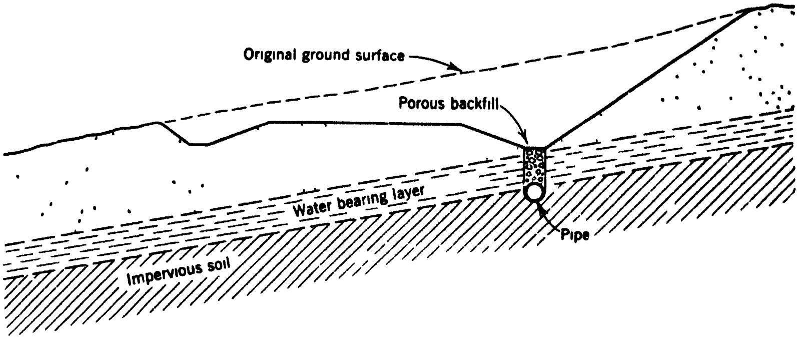


FIG 21 Drainage of inclined seepage plane

Use of Underdrainage. Free water in subgrades occurs either as flow through a permeable soil overlying an inclined layer of impervious soil or as storage water having a level water table. The first condition occurs in areas where the soil layers are inclined as in rolling or hilly territory, and the water may often flow under a head. Inclined seepage often is observed in cuts where the excavation has cut through a water-bearing layer of soil. The second condition is found in level areas and may occur near swamps or other bodies of water or in areas of considerable rainfall where the outlet is high. Capillary rise through the overlying soil may occur in both cases.

Inclined seepage may be intercepted by placing a pipe drain in the path of the flowing water. A typical interception of inclined seepage which flows across a road is illustrated in Fig. 21. It is extremely important that the bottom of the drain extend into the impervious layer and that the trench above the pipe be backfilled with granular material. *If the surface water carries silt, it is essential that the top of the back-filled trench be sealed with impervious material.* Inclined seepage may flow parallel to the axis of the road, and it then is necessary to construct a transverse pipe drain across the road at a point where the water can be intercepted without damage above the drain location; side drains also are usually necessary to carry off the water.

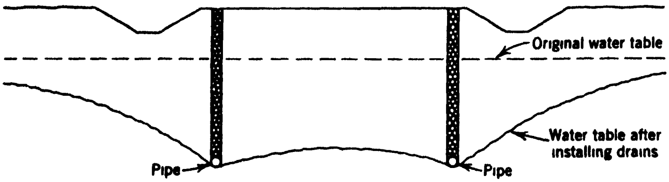


FIG 22. Lowering of level water table by underdrainage

The drainage of level water-table areas depends upon the availability of outlets and is usually accomplished by placing two pipe drains, one on either side of the road, as shown in Fig 22. Sometimes one pipe drain is placed under the center of the roadway, but it is not as effective as two side drains and requires tearing up the roadway if any repairs become necessary.

CULVERTS

A culvert is a drain or conduit for carrying surface water beneath a roadway; it differs from a bridge, which is a structure for carrying a roadway over a watercourse or a ravine. Sometimes a culvert is used in place of a bridge across narrow streams or across wide, shallow streams; multiple-opening culverts frequently are used in the latter cases. Bridges also are defined as structures having separate substructure and superstructure, whereas the substructure and superstructure are combined in culverts; this differentiation is not entirely true, because a metal arch culvert may be constructed on a concrete foundation, and the substructure and superstructure may be partially combined in a reinforced-concrete rigid-frame bridge. Some highway organizations differentiate between culverts and bridges on the basis of the span length, 6 to 12 ft being commonly taken as the dividing line.

Numerous materials are used for culverts, and shapes of openings are various.

Pipe Culverts. Vitrified clay sewer pipe, concrete sewer and culvert pipe, cast-iron water pipe, corrugated cast-iron pipe, and corrugated sheet metal pipe are used for pipe culverts.

Clay and Concrete Pipe Culverts. Vitrified clay sewer pipe and concrete sewer pipe conforming to the requirements of the American Society for Testing Materials usually is specified for culvert purposes (in the larger sizes, this calls for the class of clay sewer pipe known to the trade as "double-strength" pipe). A standard specification for reinforced-concrete culvert pipe has been adopted by the American Society for Testing Materials; concrete culvert pipe usually is made with tongue-and-groove ends rather than bell-and-spigot ends, like sewer pipe. The joints of clay and concrete pipe culverts are generally filled with Portland cement mortar.

Cast-Iron Pipe Culverts. Cast-iron water pipe with bell-and-spigot joints, lock-joint, and corrugated cast-iron pipe are used for culvert purposes and possess the advantages of long life and strength.

Riveted Metal Pipe Culverts. Corrugated sheet metal culvert pipe is made from five grades of base metal known as pure iron, copper-bearing pure iron, copper iron, copper-molybdenum iron, and copper steel. The manufacturers of pipe made from the various grades of iron claim that the purity of the iron or the alloying ingredients increases the resistance of the iron to corrosion; opinion on this subject is divided, however.

Riveted sheet metal culvert pipe is manufactured in sizes up to 72 or 84 in. in diameter and in lengths which are in multiples of 2 ft; the maximum length depends upon the method of transportation of the pipe from the fabricating plant to destination; railroads will accept lengths of the small diameters up to 22 or 24 ft but require shorter lengths as the diameters increase.

Corrugated metal pipe intended for locations where it may be subject to erosion by sand and gravel carried by swiftly moving water can be obtained with an asphalt covering over one-half of the interior surface.

Sections of riveted corrugated metal pipe are joined together by bands of the corrugated metal which are held together by bolts. Connecting bands are not less than 7 in. wide for 30-in. and less diameters, 12 in. wide for 36- to 48-in. diameters, and 24 in. wide for diameters above 48 in.

Bolted Metal Culverts. Corrugated metal plates of heavy gauges are used to fabricate circular or semicircular culverts on the job for diameters from about 90 to 150 in. and for arch spans up to about 30 ft. These plates are manufactured by the makers of corrugated sheet metal for riveted pipes and may be obtained in the various base metals. The plates are held together by galvanized bolts having specially shaped heads or washers to fit the corrugations. Galvanized plates, to which the curved plates may be attached, may also be obtained to serve as floors and aprons for arch culverts. More frequently, however, arch barrels are supported on concrete pedestal abutments in the tops of which galvanized angles or special channels are embedded or attached by means of bolts to hold the edges of the metal plates. This type of culvert is quite expensive in the larger sizes but possesses the distinct advantage of being rapidly installed. One of its applications for circular shapes is in locations where foundation conditions for small bridges do not furnish high supporting strength for abutments; it is not advisable, of course, to undertake such construction under extremely unfavorable conditions, but it offers a practical means of carrying traffic over many inferior narrow stream beds, particularly on light-traffic roads.

Concrete Box and Arch Culverts. Concrete box culverts are extensively used by some highway organizations for spans of 3 to 8 ft. They have reinforced tops, and the sides and bottoms are either plain or reinforced Concrete arch culverts are generally used under high fills because they are more economical than box culverts of equal strength. Concrete box and arch culverts are adaptable to new road construction where time is not an extremely important factor, but they are not so adaptable to replacements of existing culverts or new installations on roads over which traffic must be maintained, on account of the time and cost involved in formwork and time necessary for curing the concrete. The use of high-early-strength Portland cement overcomes this disadvantage to a large extent.

Masonry Arches. Stone masonry arch culverts formerly were used to a large extent where good stone was available at low cost but are rarely constructed now on account of their high cost compared with concrete or heavy corrugated metal. If desired for appearance, a concrete or metal barrel may be used with stone masonry headwalls.

Timber Culverts. Timber is not employed very extensively for culverts, although it is used to some extent in areas where its cost is low; timber culverts are usually of the box type, and the timber is creosoted.

Length of Culverts. The length of culverts should never be less than the distance between the outside shoulder lines and will depend to a large extent upon the height of earth above the culvert. Generally, it is more economical to lengthen the barrel of small culverts than to build large headwalls, but the cost of each can be calculated in any case to determine the most economical design.

Headwalls. Concrete headwalls are used to the greatest extent for pipe, concrete, and metal arch culverts. Stone masonry headwalls are frequently employed to enhance the appearance of culverts, particularly with pipe and arch culverts. Metal headwalls are sometimes used with riveted corrugated metal pipe culverts and also with circular bolted metal pipe culverts; they possess the advantages of quick installation and of being salvaged when replacement is necessary or when the location of a culvert is to be changed.

Headwalls frequently are necessary to prevent erosion at the ends of culverts, but they are used in many locations largely for the sake of appearance. Headwalls form a large portion of the cost of culverts and may be eliminated in many locations by using riprap or by lengthening the barrel of pipe culverts on both ends or on the discharge end.

Paving of the channel at the discharge end of pipe culverts is often necessary to prevent excessive erosion; a concrete apron or riprap usually will serve the purpose.

Area and Shape of Waterway Opening. The area and shape of the waterway opening of culverts depend upon the volume and the rate of flow of the water, upon the shape of the natural channel, upon the height available, and other physical characteristics of the site. Visual evidence at the site, such as previous high-water marks and observation of the flow during periods of high water, supplemented by information from local residents, will often be sufficient to determine the necessary area of the opening.

Several formulas, developed to serve as guides in determining the area of waterway openings, are useful where definite information relative to the volume of water is not available. These formulas may be conveniently classified into two groups: in the first group the calculation is based on the drainage area; in the second group the formulas take into consideration rainfall data in addition to the drainage area. Talbot's formula is the most widely used in the first group; it is:

$$a = c\sqrt[4]{A^3} \tag{5}$$

in which a = the area of the waterway opening, in square feet.
A = the drainage area, in acres.

c = a constant depending upon topographical and climatic conditions.

The constant c is usually selected as the result of experience in a particular locality. Average values of c are recommended as follows:

$c = \frac{2}{3}$ to 1 for mountainous or very hilly country.
$c = \frac{1}{3}$ for rolling country.
$c = \frac{1}{6}$ to $\frac{1}{5}$ for flat country.

The Burkli-Ziegler formula is representative of formulas that take runoff into consideration:

$$Q = cAR\sqrt[4]{\frac{S}{A}} \tag{6}$$

in which Q = the quantity of water that reaches the culvert, in cubic feet per second.
R = the average rainfall during the heaviest fall, in cubic feet per second per acre (1 in. of rainfall per hour may be taken as equal to 1 cu ft per sec per acre).
S = the average slope of the ground comprising the drainage area in feet per 1000 ft.
A = the drainage area, in acres.
c = a constant which depends upon topographical and climatic conditions.

The following average values for c are commonly recommended:

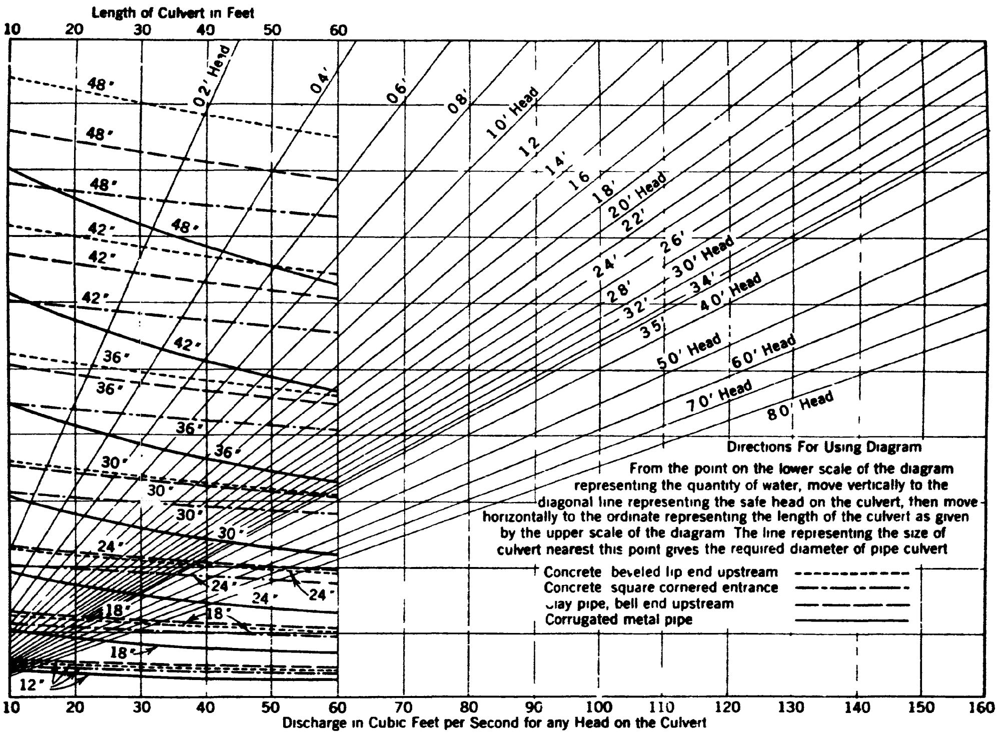
c = 0.20 to 0.25 for rural areas.
c = 0.30 for municipal areas with macadam streets.
c = 0.625 for municipal residential areas with paved streets and other areas not fully built up.
c = 0.75 for municipal areas with fully built up business blocks and paved streets.

The sizes of pipe and box culverts required for various quantities of water may be determined from the diagrams in Figs. 23 and 24.

BRIDGES

Bridges carry roadways over streams, large depressions in the road (such as ravines), railroads (railroad grade separations), and other highways (highway grade separations). Bridge structures frequently are described as consisting of a substructure, or foundation, and a superstructure.

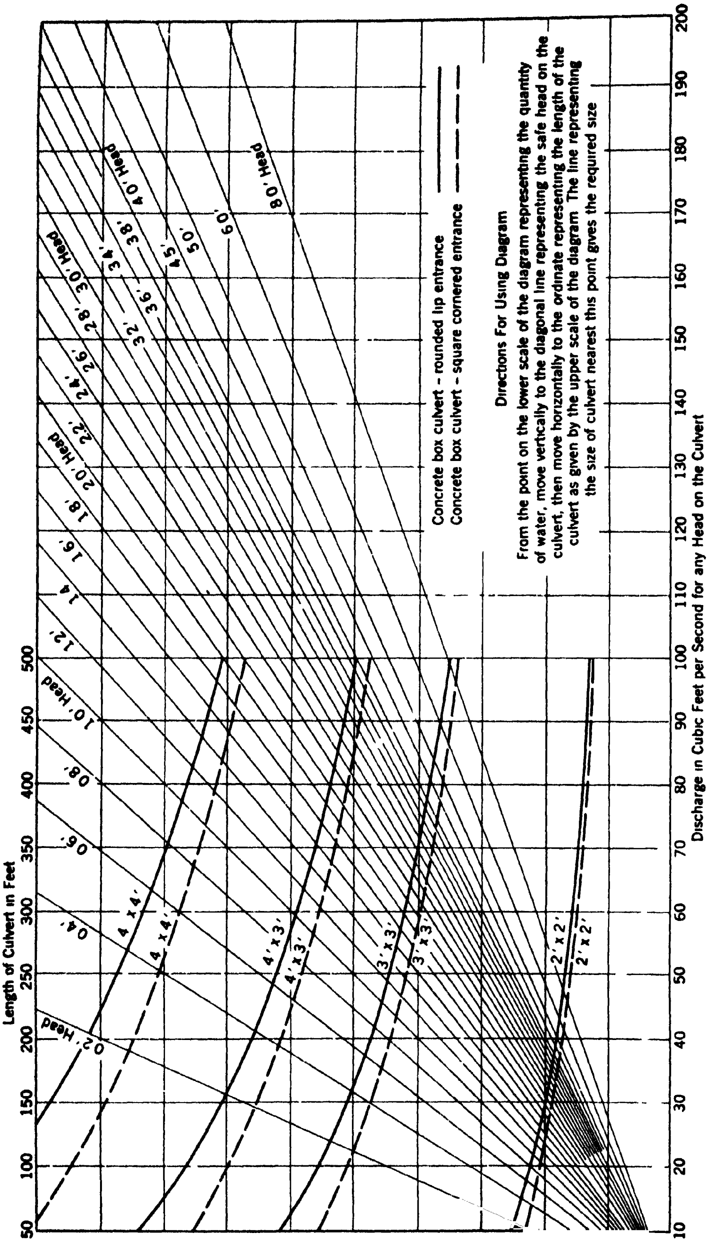
Substructures. The principal bridge substructures are abutments, piers, and trestle bents. The several types of abutments include concrete gravity abutments, reinforced-concrete abutments, caissons, and pile bent abutments. Concrete gravity abutments are seldom used on



Directions For Using Diagram

From the point on the lower scale of the diagram representing the quantity of water, move vertically to the diagonal line representing the safe head on the culvert, then move horizontally to the ordinate representing the length of the culvert as given by the upper scale of the diagram. The line representing the size of culvert nearest this point gives the required diameter of pipe culvert.

Fig. 23. Diagram for determining the size of a pipe culvert required to carry a known quantity of water under a given head for lengths of 10 to 60 ft. Culverts with straight endwall entrances. (*University of Iowa Engineering Studies, Bul. 1.*)



Directions For Using Diagram

From the point on the lower scale of the diagram representing the quantity of water, move vertically to the diagonal line representing the safe head on the culvert, then move horizontally to the ordinate representing the length of the culvert as given by the upper scale of the diagram The line representing the size of culvert nearest this point gives the required size

FIG. 24. Diagram for determining the size of a box culvert required to carry a known quantity of water under a given head for lengths of 50 to 500 ft. Culverts with straight endwall entrances. (*University of Iowa Engineering Studies, Bul. 1.*)

account of their high cost in comparison with reinforced-concrete abutments. The latter usually are of the cantilever type, but counterfort abutments are frequently used for high foundations. Caissons are a special type of abutment or pier which are suitable where great depths of soft soil are encountered; they consist of circular or rectangular sections of steel or concrete which are built up and lowered into place or sunk by weighting and excavating. Since bridge foundations may easily represent the major portion of the cost of bridges, particularly where soils are unfavorable, economy of construction demands very careful attention to foundation design. Pile bent abutments offer many opportunities for effecting economies in the construction of small bridges on light- to medium-traffic roads; they consist of timber or steel piles supporting a concrete bridge seat and are generally located at or near the top of the stream banks. They are particularly adaptable to I-beam bridges; protection against scour often is secured by riprapping the stream banks adjacent to the abutments.

Timber Bridges. Creosoted timber bridges are built in locations not too far from sources of supply of timber on account of their low cost and the rapidity with which they may be constructed. The beam type of timber bridge is constructed almost to the total exclusion of other types, and steel beams and concrete floors are commonly employed to increase the useful life.

Concrete Slab Bridges. Concrete slab bridges consist of a reinforced-concrete slab supported directly on the abutments or on abutments and piers, although multiple-span slab bridges are seldom built. They are economical for short spans only, the limiting length seldom exceeding 16 ft.

Concrete Beam and Girder Bridges. Concrete beam bridges of the T-beam type occupy an intermediate place between the concrete slab and concrete girder types.

Concrete girder bridges, which may be either of the deck or through type of design, consist of main longitudinal beams (called girders), transverse beams, and the floor slab. The transverse beams sometimes are omitted in narrow through girders (18- to 20-ft roadway widths) but are an essential feature of bridges designed for safe widths and heavy loads. The through type of girder is objectionable on account of the appearance of the heavy girders at the sides, but the deck girder presents no difficulties in design to obtain a satisfactory appearance. Concrete girder bridges are usually built in span lengths of 20 to 50 ft, although some longer spans have been constructed.

I-Beam Bridges. I-beam bridges are very commonly used for spans varying from 10 to 60 ft, the span length being controlled by the maximum size of beam which can be rolled. The low cost of these bridges, together with the fact that shoring is unnecessary, has made them extremely popular; their principal objection is the need for periodical painting of the I-beams.

Rigid-Frame Bridges. Rigid-frame, reinforced-concrete bridges, on account of their economy and adaptability to various architectural treatments, are favored for many locations on rural highways and municipal streets.

Plate-Girder Bridges. Plate-girder bridges may be of either the through or deck types and are adaptable to long and deep crossings. In general, they are limited to spans of 60 to 100 ft.

Steel Truss Bridges. Steel truss bridges are of three types, the low steel truss without overhead bracing, the high steel truss with overhead lateral bracing, and the deck truss which carries the roadway on the top chord. The low steel truss has a rather limited application to span lengths of about 60 to 100 ft; frequently it is used in place of plate girder bridges where insufficient headroom is available for the deck girder type of design. The deck truss is used on high bridges where the elimination of the overhead steel is desirable on account of appearance.

The high steel truss is one of the most frequently used types of bridges for long spans, is economical, and, when care is exercised in design, presents a satisfactory appearance in many locations. It is economical for span lengths as low as 100 ft, and, with cantilever-truss supports, is used for extremely long spans. This type of bridge is adaptable to multiple-lane roadways and frequently serves combination railway and highway purposes.

Arch Bridges. Arch bridges, of reinforced concrete or steel, are widely used on account of their pleasing appearance. For long spans they frequently are less costly than other types of bridges. Two conditions are essential for arch construction—good foundations and sufficient headroom.

Arch bridges are built in span lengths varying from a few feet to several hundred feet. Notable examples of long steel arch spans are the Bayonne Bridge in New Jersey, having a span of 1652 ft, and the Sydney Harbor Bridge in New South Wales, having a span of 1650 ft. Long-span concrete arch bridges have been constructed in Europe, the most outstanding example being the Albert Louppe Bridge over the Elorne near Brest, France, consisting of three arch spans, each

612 ft from center to center of the supports, and designed by the eminent French engineer, E. Freyssinet.

A type of bridge known as a cantilever arch bridge sometimes is constructed when foundation conditions are unsatisfactory for a structural arch; this type of bridge is not an arch bridge except in shape and consists of a cantilever truss, shaped like half of two arches and producing the architectural effect of an arch on multiple-span bridges, usually of comparatively short span length. An outstanding example of this type of bridge is the vehicular bridge connecting the city of Detroit with Belle Isle in the Detroit River; the foundations of this bridge were constructed on gravelly soil which was not considered satisfactory for true arch construction.

Suspension Bridges. Suspension bridges have been used on rural highways only in rare cases because of their high cost. In general, they are adaptable only to long spans and are resorted to in locations where the construction of piers for supporting other bridge types of shorter spans is impracticable.

Movable Bridges. Movable bridges are used on highways and streams crossing navigable waters. Swing bridges, supported on a central pier on which they rotate, are slow and unwieldy and, unless very carefully protected, constitute a dangerous hazard; the high steel truss is most widely used for this type of movable bridge. Bascule bridges are rapid in operation, are adaptable to wide roadways, have minimum interference with water traffic, and, when properly designed, have a pleasing appearance; they may consist of a single leaf or two leaves, meeting in the center, depending upon the span length. Lift bridges are raised vertically and are adaptable in many locations because of limited water clearance; they are much less common than bascule bridges.

Bridge Floors. Bridge floors are of many types. Concrete floors are extensively used on concrete beam and concrete girder bridges, on steel beam bridges, on steel and concrete arch bridges, on steel truss and plate girder bridges, and on concrete slab and rigid-frame bridges. Bituminous and block types of wearing surfaces often are placed on concrete slab foundations, particularly under extremely heavy traffic conditions, and may be renewed without long periods of interference with traffic. Timber plank floors are suitable for timber and steel bridges but should be restricted to light-traffic roads and streets; creosoted timber should be used for subfloors but is not generally adapted to floors for wearing surfaces unless steel traffic plates are employed. Asphalt plank floors are quite common but have given trouble from slipperiness and therefore are not very satisfactory on bridge floors which have horizontal grades.

Steel plate floors are found to a limited extent on steel bridges and are secured to the steel floor beams by welding. The economy of this type of floor is questionable in most instances.

Figure 25 illustrates a steel grid which is used for bridge floors and which is composed of steel T-sections welded together and reinforced with cross-members welded or pressed into slots; it generally is fabricated into sections 4 ft in width. The grid is plug-welded to the floor stringers and then is filled with a rich, stiff concrete which should be vibrated into place. The concrete sometimes is placed flush with the surface of the grid, but a better appearance can be secured and suitable crown obtained by placing from $\frac{3}{4}$ to $1\frac{1}{2}$ in of concrete over the top of the grid. This type of floor is rather expensive, but the author has found it particularly adaptable to the reconstruction of old bridges which were of too light construction to support a reinforced-concrete slab without seriously decreasing the safe carrying capacity of the bridge.

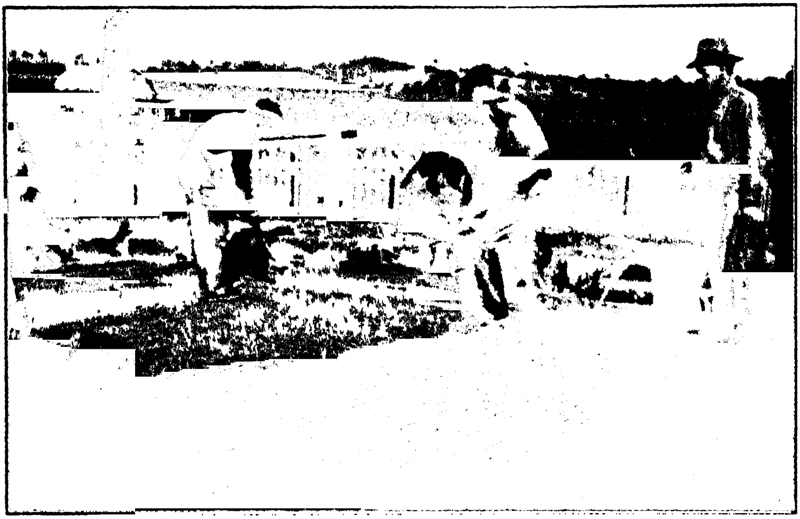


Fig. 25 Vibrating concrete into place on a steel grid floor

Steel bridge floors on steel bridges are sometimes of open-mesh types in order to effect savings in the weight of floor systems. Such floors also possess the advantages of having a non-skid surface and of being self-draining and self-cleaning; the latter features are of importance in regions where snow is prevalent.

Bridge Rails, Curbs, and Sidewalks. Railings, curbs, and sidewalks are important features of bridges from the viewpoints of utility, safety, cost, and appearance.

Concrete masonry railings, if properly designed, are satisfactory on concrete bridges of short span. The Illinois Division of Highways has constructed some bridge railings of combination concrete and brick masonry which are pleasing in appearance as well as substantial. Heavy wrought-iron pipe rails, set in concrete posts, are very satisfactory for small concrete or steel beam bridges which are not high above the water.

Precast concrete spindle railings are used extensively on concrete bridges; when properly designed and constructed, they present a very pleasing appearance. Precast concrete planks sometimes are used for low short-span concrete bridges.

Various types of steel railings are used on both steel and concrete bridges. The steel lattice rail common on steel truss bridges is a satisfactory railing if made of sufficiently heavy material and so located that it is not easily damaged. Structural steel railings with vertical members combined with a pleasing top treatment are among the most satisfactory types for deck bridges of either concrete or steel construction.

Wooden railings are used on timber bridges and, to a limited extent, on small steel bridges having timber floors.

Curbs are necessary features of bridges to protect railings and structural units above the floor level. Timber curbing is used on timber bridges or on bridges having timber floors; painted curbs are desirable and are mounted on scupper blocks to permit drainage. Concrete curbs provided with drainage openings are most widely used; they should be very carefully designed so that vehicle wheels will not tend to climb over them or hubcaps catch on the upper edges of the curbs.

Sidewalks are an important part of bridges on municipal streets and in some rural areas. Concrete sidewalks are desirable, and it is best for them to be definitely separated from the roadway by means of railings.

Bridge Clearances. The width of bridges should be sufficient for the passage of two vehicles; the American Association of State Highway Officials recommends a minimum roadway width of 18 ft. Short bridges should be at least 6 ft wider than the width of a two-lane roadway, and often they are wider. A minimum vertical clearance of 14 ft is recommended for bridges with overhead bracing by the American Association of State Highway Officials.

Bridge Location and Approaches. The speed and volume of rural highway traffic require care in the location of bridges and the design of the approaches. Locations at or near sharp curves should be avoided if at all possible, and steep gradients are undesirable. Painting the superstructure of steel bridges a light color is very helpful, and signs or outline markings having reflector sign buttons are useful for bridges at which the portals are somewhat obscured. Guard-rail approaches painted white or light yellow are advisable, and end posts or ends of railings of long bridges should be protected by heavy posts to prevent damage by vehicles.

Determination of Span and Waterway Opening. The physical contour of the ground will determine the length and height of bridges to be built across ravines without large flowage of water and across waterways with banks which are much higher than high water. Ordinary watercourses generally flow in well-defined channels, and high-water elevation usually can be determined from marks on the channel, on trees and poles, and from recorded observations or from information furnished by local residents. After the height of the waterway opening is determined, the span length usually is fixed by the ground contour. Small watercourses at times are very deceptive, and a thorough examination of all possible evidence bearing on the height and volume of high water often will repay itself many times; some watercourses naturally carry a large amount of debris at some water stages, and the design of the waterway opening to prevent clogging may be important. The determination of the drainage area of small streams may be desirable, but ordinarily this determination is unnecessary if a thorough study of water conditions at the site is made.

Reconstruction of Old Bridges. Many opportunities of effecting economies often exist in the reconstruction of old bridges. Large numbers of steel bridges were built in some sections of the country prior to or early in the motor-vehicle period, and though most of these structures on main routes have been replaced or rebuilt, many of them still remain on purely local roads and often may be reconstructed at much less expense than by removing or replacing. Steel bridges offer the greatest opportunities; floor beams can be strengthened by welding plates on the bottoms (or both tops or bottoms), stringers and cover plates can be replaced, and numerous other parts of the structure can be improved. The construction of new floor systems often presents difficulties in regard to load capacities, but some of the light-weight floorings which have been described often will make reconstruction feasible.

Maintenance of Highway Bridges. The need for thorough maintenance of bridges should be apparent, but no item of road maintenance is more neglected in some areas. One of the most important items of bridge maintenance is the periodical cleaning of bridge seats to prevent the corrosion of connections and interference with expansion and contraction. Steel bridges or exposed steel bridge parts should be repainted at intervals after thorough cleaning; under favorable conditions steel bridges will require repainting about every six to eight years. Drainage of floors and of abutments should be kept open, and minor repairs should be made promptly.

Steel bridges which have been neglected for many years are difficult to clean for repainting, and many times, when heavy rust coatings are present, it is questionable whether the expense of thorough cleaning and repainting is justified unless the structure is large. The only satisfactory cleaning in such cases is secured by means of a sandblast; it is essential that the prime coat of paint follow immediately after sandblasting.

LEGAL LIMITATIONS RELATIVE TO HIGHWAY DRAINAGE

Drainage of Surface Waters. The highway engineer engaged in the improvement of a rural highway is always confronted with the problem of drainage, particularly with reference to surface waters which fall directly on the highway or which reach the highway from adjacent property. Laws relating to the drainage of surface waters and in regard to natural watercourses and lakes and ponds vary in the different states, and the highway engineer and highway official must be familiar with the laws in his state in order to provide for effective drainage and to avoid disputes with the owners of adjacent and other lands affected by changes in drainage.

Additional Right-of-Way. The construction of bridges, ditches, and drains frequently involves the permanent or temporary use of land outside the limits of the highway right-of-way, and arrangements with property owners who are involved should always be made prior to construction.

Bridges over Navigable Waterways. The construction of bridges over navigable waterways in the United States requires the approval of plans by the U. S. War Department.

Special State Requirements. Some states, by law, require bridges and culverts to meet specific requirements relative to load capacity, width, and other features. Some state laws also require highway departments to erect warning signs at or near bridges which are narrow, have low clearances, or have carrying capacities below a definite standard.

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CHAPTER IV

GENERAL FEATURES OF HIGHWAY DESIGN

The design of highways and streets embraces various general features of width and alignment, grades, intersections, grade separations, drainage, and miscellaneous structures. The design of drainage systems and structures was discussed in Chapter III.

WIDTH AND ALIGNMENT

The selection of the width of highways and streets involves the determination of the width and number of traffic lanes, the width of shoulders, the width of parking strips, the width necessary for ditches, gutters, drains, and public utility services, the width of sidewalks, and the width of dividing strips.

Traffic Lanes. Two ordinary private passenger vehicles traveling at slow or moderate speeds can pass each other safely on a roadway width of 16 ft; high speeds and larger vehicles require greater widths. Most state laws limit the gross width of motor vehicles to 8 ft, and a width of 9 ft, which provides at least 2-ft clearance between passing trucks or busses of maximum width, therefore may be considered as the minimum desirable width of a traffic lane, and is suitable for roads and streets which carry only a small proportion of truck and bus traffic and on which traffic does not move at high speeds. Ten-foot widths are suitable for moderately heavy-traffic roads and streets; 11-ft widths are used for heavily traveled rural highways and streets on which the traffic moves at high speeds. Increasing the width of traffic lanes beyond 11 ft has the effect, on multiple-lane roadways, of introducing an additional lane for each two intended lanes and creating the hazard of narrow lanes

Theoretical Capacity of Traffic Lanes. The maximum capacity of one lane of traffic which is subject to no interruptions or interference and on which all vehicles travel at the same constant speed may be expressed by the general formula

$$N = \frac{5280\,V}{C + L} \tag{7}$$

in which N = the number of vehicles per hour.
V = the speed of the vehicles, in miles per hour.
C = the clearance between each two consecutive vehicles, in feet.
L = the length of the vehicles, in feet.

The average length of vehicles is assumed to be 15 ft. The clearance of vehicles should be equal to the stopping distance of a vehicle in case an obstacle to travel is encountered. The stopping distance of a vehicle is dependent upon the sight reaction time of the driver, or the time for seeing the obstacle, upon the brake reaction time of the driver or the time required for applying the brakes, and the brake stopping distance. The sight reaction time varies generally between 0.5 and 3.0 seconds, and the brake reaction time from 0.2 to 1.5 seconds. A total reaction time of 4 seconds is sometimes assumed for design purposes. The brake stopping distance is a function of the speed of the vehicle, the condition of the tires, the design and condition of the brakes, the type and condition of the roadway surface, and the gradient of the roadway; it often is described in terms of the deceleration of the vehicle, as follows:

$$S = \frac{Vt}{1.5} + \frac{V^2}{4.5a} \tag{8}$$

in which S = the sight distance (assumed equal to the stopping distance), in feet.
V = the speed of the vehicle, at the time of observing the obstacle, in miles per hour (1 mi per hr may be assumed as 1.5 ft per sec).
t = the total reaction time of the driver, in seconds.
a = the rate of deceleration of the vehicle, in feet per second per second.

The rate of deceleration varies from about 20 to 5 ft per sec per sec. A value of 12 ft per sec per sec was found by Professor Moyer on dry surfaces under ordinary emergency conditions. From 8 to 9 ft per sec per sec is desirable for comfort and has been recommended for design purposes. A value of 5 ft per sec per sec, or less, must be assumed for icy road conditions. Numerous formulas have been proposed for determining theoretical traffic capacities, based on different assumptions in determining clearance; three of these formulas are as follows:

Johnson formula $$N = \frac{5280V}{0.5V^{1.3} + 15} \tag{9}$$

Dougherty formula $$N = \frac{5280V}{0.0556V^2 + 0.73V + 15} \tag{10}$$

Greenshields formula $$N = \frac{5280V}{1.1V + 20} \tag{11}$$

Free-Moving Traffic Capacity. The actual capacity of a traffic lane is less than its theoretical capacity because of varying lengths of vehicles, varying speeds, varying safe stopping distances, and various traffic interferences. The terms "working capacity" and "free-moving capacity" of roadways are widely used to describe the maximum traffic capacity at which there is freedom from congestion. A. N. Johnson defined working capacity or free-moving capacity as the volume of traffic at which congestion occurs and defined congestion as the number of vehicles which fill the road and make turning out impracticable, this condition to last a sufficient length of time to be noticeable, the minimum amount of time being 1 minute. Adapting a definition by the Highway Research Board, in 1928, "the *free-moving* traffic capacity of a roadway is reached when any further increase in traffic volume, all other factors remaining constant, results in a marked decrease in traffic speed."

Traffic interferences reduce the actual capacity of roadways. These interferences are classified into four groups, as follows:

1. Cross-interference, which occurs as the result of two streams of traffic which cross each other at the same elevation or grade.
2. Marginal interference, which occurs between moving vehicles and objects or people along the outer edge of the roadway.
3 Internal interference, which occurs between vehicles moving in the same direction.
4. Medial interference, which occurs between vehicles moving in opposite directions along the central portion of the roadway.

The free-moving traffic capacity of rural highways naturally varies over a wide range and has been estimated by various investigators to be from 200 to 500 vehicles per hour per lane.

The free-moving traffic capacity of traffic lanes on city streets varies over a wider range than that of rural highways. The traffic on city streets is slower than on rural highways, which tends to raise the capacity, but the presence of streetcars, busses, pedestrians, parking spaces, and frequent intersections interferes with the movement of vehicles to a much greater extent than on rural highways. It is obviously impossible to develop precise values for various kinds of traffic interferences. Numerous investigations of street traffic have been made, and the following conclusions reported by Lewis

W. McIntyre on the basis of studies of street traffic in Pittsburgh are of interest:

1. The traffic capacity of a single lane of street in a congested city district, with interferences eliminated, is from 2000 to 2500 passenger automobiles per hour.
2. Interference by cross-traffic at grade reduces the capacity of any street approximately 50 per cent.
3. The stopping of a lane of traffic reduces the capacity of that lane 35 to 50 per cent when the stops occur close together.
4. In the absence of pedestrian control, as much as 23 per cent of the pedestrian traffic crossing the street will interfere with vehicles, thus reducing the street capacity.
5. On the average, a passenger automobile turning left may delay following traffic a length of time equal to that required for four such vehicles to go straight through.
6. A large parking lot or garage may have the effect of reducing the capacity of a main thoroughfare from 20 to 40 per cent. The extent of the reduction depends on the number of lanes of moving traffic on the street.
7. Curb parking reduces traffic capacity about 50 per cent.
8. The traffic flow of city streets is not proportional to the number of lanes.
9. Trucks reduce the capacity of a street 25 to 40 per cent below that for passenger automobiles.

Rural Roadway Widths. Rural highways usually are built with an even number of traffic lanes, two-, four-, and six-lane widths being common. Highways having an odd number of traffic lanes (such as three or five) are not considered safe by some highway authorities because vehicles traveling in opposite directions must pass overtaken vehicles in the single central lane. Under good conditions of visibility, the weight of this objection is materially diminished, and numerous three-lane pavements have indicated their safety and economy.

The determination of the number of traffic lanes on a rural highway generally is not a problem unless the traffic volume is greater than the free-moving capacity of a two-lane roadway, which is the minimum condition. It is apparent that no fixed rule can be adopted which will be generally applicable. In many locations the problem consists of building additional lanes as traffic volumes increase to the point of congestion. Traffic flow is not constant but normally varies from hour to hour, from day to day, and from month to month. Figures 26 and 27 illustrate several typical variations in traffic flow. Some highway organizations have adopted hourly traffic as a criterion for selecting the number of traffic lanes and recommend a four-lane roadway (or, in exceptional cases, a three-lane roadway) when the

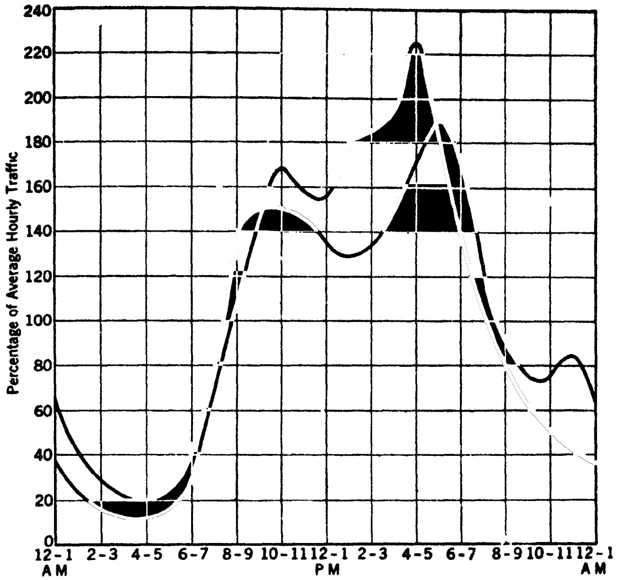


Fig 26 Diagram illustrating hourly variation in highway traffic

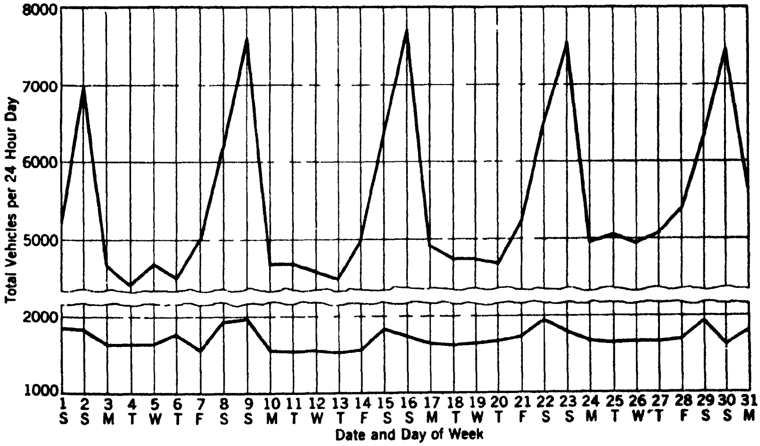


Fig 27 Daily traffic on two primary state highways

hourly traffic reaches 500 to 600 vehicles regularly during three hourly periods of weekdays The American Association of State Highway Officials recommends more than two traffic lanes when the average daily traffic exceeds 4000 vehicles.

The criterion for selection of a six-lane roadway instead of a four-lane roadway has not been generally established. The economy of building highways having eight or more lanes is questionable, and when more than four or six lanes become needed, an additional high*way generally will be more desirable.*

Additional Lanes on Hills for Slow Traffic. *Slow-moving trucks often impede traffic* on two-lane pavements in hilly locations. Where relocation or grade reduction is impracticable the construction of an additional lane on rising grades, which may result in a three- or four-lane pavement, for the use of the slow-moving vehicles, may be justified.

Street Roadway Widths. The width of street pavements should vary according to the class of street, the presence of car tracks, and provisions for parking. A width of 9 ft generally is allowed for a single-car-track area, but since car-track areas serve as traffic lanes, a minimum width of 10 ft is desirable. Parallel parking for automobiles requires at least 8 ft, and diagonal parking requires about 16 ft. Twenty-foot parking widths are desirable on industrial streets on which trucks are permitted to load and unload. Pavement widths, exclusive of car tràcks, for various classes of streets, therefore, should be approximately as follows:

Residential streets, land service only	26 to 40 ft
Residential streets, through-traffic	36 to 60 ft
Retail business streets, primarily local	40 to 60 ft
Retail business streets, through-traffic	56 to 80 ft
Industrial streets, minimum	60 ft
Arterial streets	56 to 120 ft

Width of Shoulders. The standard width of shoulders on through roads has steadily increased, and 8 to 10 ft now is favored for heavy traffic. For secondary roads 6- to 8-ft widths are desirable minimums. The width of shoulders on land-service roads is generally controlled by placing a minimum limit on the overall width of the surface and shoulders. Some counties have adopted minimum overall widths of 20 to 22 ft for improved land-service roads, but the widths should be greater in rolling country where fills and side ditches (rather than shallow gutters) are necessary; 24 to 26 ft are desirable minimum widths under such conditions.

Side Slopes. Shoulder slopes should be somewhat greater than roadway crowns, and a slope of ¾ to 1 in. per ft is representative of present practice. Side slopes should not exceed the angle necessary for the stability of the soil against sliding; slopes of 1½ horizontal to 1 vertical are maximum slopes for the inside and outside of ditches for ordinary soils. Slopes of ¼:1 or even vertical faces are satisfactory in many rock cuts and in loess soils. V-bottom ditches with easy slopes are desirable from the viewpoints of cleaning, machine mowing, and snow removal.

Sidewalks. The width of sidewalks should be in proportion to the pedestrian traffic which will use them, with a minimum of 5 ft. The sidewalk on business streets usually covers the entire width from the building lines to the curb; on residential streets a space is left between the sidewalk and the curb to present a pleasing appearance, and a space of 1 ft sometimes is left between the property line and the edge of the sidewalk to facilitate the installation and care of fences and hedges. It is almost universal practice to provide a minimum transverse slope of 2 per cent from property lines to curbs in order to facilitate drainage; this slope may be increased to 6 or even 10 per cent at intersections on streets having steep grades.

Sidewalks and pathways are provided on many rural highways in areas having large amounts of pedestrian traffic.

Dividing Strips. The critical condition with respect to the width of dividing strips on multiple-lane roads and streets occurs at intersections where crossing vehicles may be compelled to wait for passing traffic. At least 10- to 12-ft widths are desirable, and, depending on the volume of crossing traffic and the total width of the road or street, widths up to 30 or 40 ft may be desirable. Widths of dividing strips between intersections may be as little as 4 ft if ample curb protection is provided, although 8 ft is a desirable width to enable the strip to be used as an emergency parking lane. Approaches to intersections having widened dividing strips should, of course, be made in distances which are sufficient to provide safe curvature of the roadway. Some highway and street departments employ light-reflecting curbs along the sides of dividing strips to improve visibility, particularly at night. These curbs have a finish coat of mortar made with white Portland cement and are scored on their upper face. (See Fig. 28.)

Traffic Separators. Traffic separators are frequently installed between opposing traffic streams where dividing strips are not feasible because of financial or other considerations. The separator shown in Fig. 29, developed from basic studies by the Bureau for Street Traffic Research at Yale University and from driving tests, is intended to prevent an automobile from either hurdling the separator or being hurled back out of control into the traffic stream. Traffic separators are well adapted to municipal street thoroughfares but are considered undesirable for roadways on which speeds above 50 or 55 mi per hr are prevalent.

Width of Right-of-Way. The widths of the right-of-way, the roadway or street pavement, the sidewalks, shoulders, and roadsides

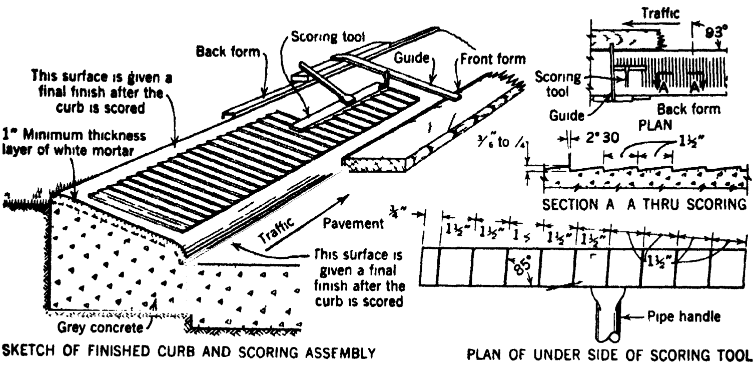
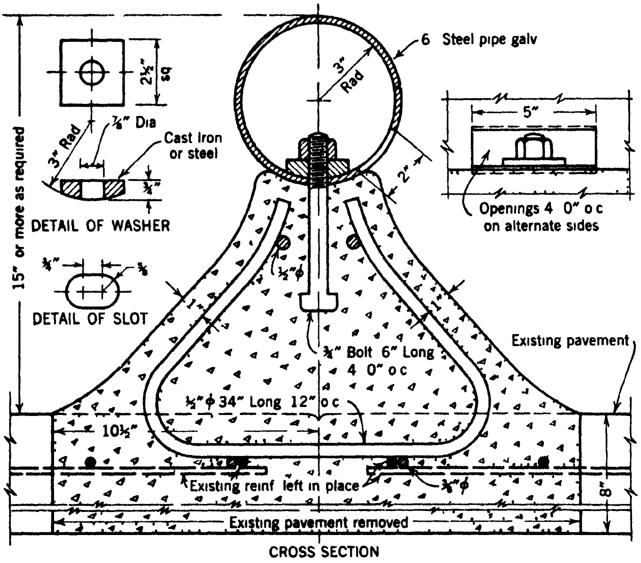

Back form
Scoring tool
Guide
Front form
This surface is given a final finish after the curb is scored
1" Minimum thickness layer of white mortar
Traffic
Pavement
This surface is given a final finish after the curb is scored
Grey concrete
SKETCH OF FINISHED CURB AND SCORING ASSEMBLY
Traffic
93°
Scoring tool
Guide
Back form
PLAN
A
A
3/16 to 1/4
2°30
1½"
SECTION A A THRU SCORING
¾"
1½"
1½"
1½"
1½"
1½"
85°
1½"
Pipe handle
PLAN OF UNDER SIDE OF SCORING TOOL

FIG 28 White reflecting curb and detail of scoring tool (*New Jersey State Highway Department*)



Courtesy Portland Cement Assoc

FIG 29 Traffic separator installed on Interborough Parkway in Brooklyn

are intimately related The width of the right-of-way required for a rural highway depends upon the width of the roadway and shoulders, the size of ditches and gutters, the height of embankments and depth of excavations, the presence of car tracks and sidewalks, the required width for bridges and culverts, and other physical features. Sufficient right-of-way should be set aside on important through-highways in the early stages of their development to provide for future improvements, such as increases in roadway widths, the addition of sidewalks, the installation of lighting equipment and roadside improvements, and to prevent encroachments which will interfere with visibility and mar the appearance of the highway. The elimination of ditch sections in cuts by means of lip curbs or paved gutters reduces the width required for carrying outside slopes. The legal width of right-of-way varies in the different states; 66-ft (4-rod) widths are common in many sections of the Middle West, whereas in other sections 40- and 50-ft widths are not uncommon. The minimum width of a highway which serves only as an entrance and outlet for land is probably about 30 ft when the height of fills does not exceed 2 to 3 ft; in most locations, however, a minimum width of 60 to 66 ft is desirable. The width of right-of-way along a rural highway need not be uniform, and it is common practice to obtain additional widths for wide cuts and fills, for curves, for building bridges and culverts beyond normal right-of-way boundaries, and for similar purposes. The following right-of-way widths represent present practice:

Light-traffic land-service roads	30 to 66 ft
Secondary county highways, moderate-traffic primary county highways and secondary state highways	66 to 100 ft
Heavy-traffic two-lane highways	80 to 120 ft
Four-lane highways	100 to 200 ft
Six- and eight-lane highways	120 to 300 ft

Several typical right-of-way cross-sections of rural roads are shown in Fig. 30.

Right-of-way widths for urban streets vary greatly according to the class of street and the early plans of the municipalities. A minimum width of 66 ft is desirable, but widths as low as 50 ft are common. Limitations on pavement widths have already been described, and the widths of right-of-way may be determined by providing the necessary widths for sidewalk areas, central dividing strips, and car-track areas.

Crowns. A transverse slope or crown across a pavement is necessary in order to carry off surface water quickly. The upper boundary of the pavement or wearing course cross-section may consist of a circular or parabolic arc, of two intersecting straight lines or curved lines, or of a combination of a circular or parabolic arc in the central portion and tangents in the outer portions. In most cases there is but

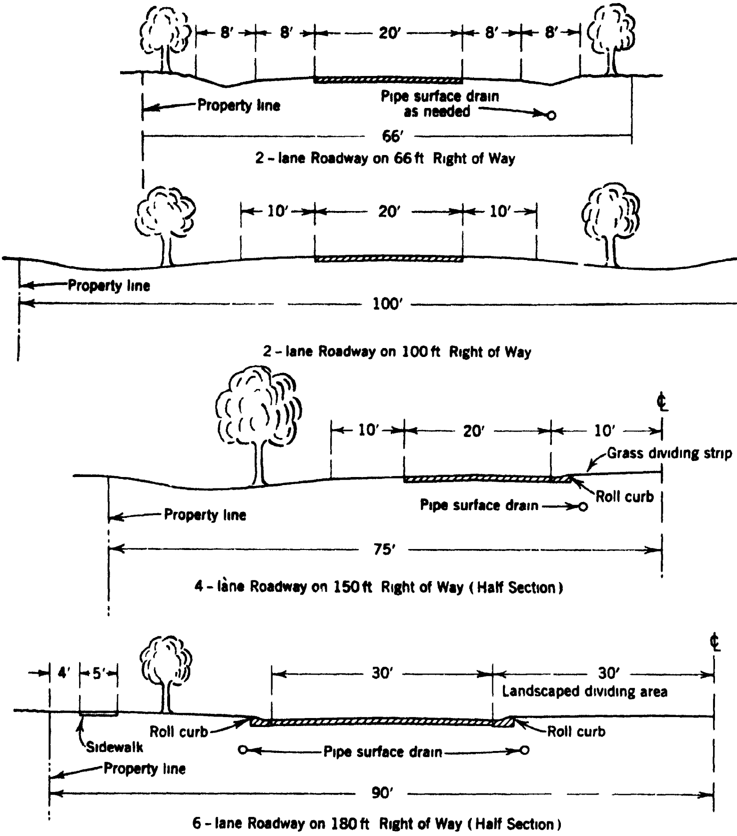


FIG 30 Right-of-way cross-sections for rural roads

little difference between a circular or parabolic arc having the same vertical rise, and the parabolic arc is chosen on account of the simplicity of calculating ordinates from a tangent in order to set grades or construct a template. The formula for calculating these ordinates is

$$y = \frac{4cx^2}{w^2} \tag{12}$$

in which y = the vertical distance of a given point on the parabola below a horizontal tangent at the center line, in inches.
x = the horizontal distance of the given point on the parabola from the center line, in feet.
c = the total crown of the cross-section, in inches.
w = the width of the cross-section, in feet.

The crown of a wearing surface or pavement should be established in accordance with the smoothness and regularity of the surface. Crown usually is expressed in terms of the rate of the crown in inches per foot; for parabolic crowns, the rate of crown, therefore, is the average rise per foot from the edge to the center line. Crowns of ⅜ to ¾ in. per ft are used on earth roads and of ¼ to ½ in. per ft for gravel roads, surface-treated macadam, and similar surfaces. Crowns of ⅛ to ¼ in. per ft are satisfactory for well-constructed high-type pavements. Some highway and municipal engineering departments establish crowns in terms of the total rise and vary the total crown according to several ranges in width.

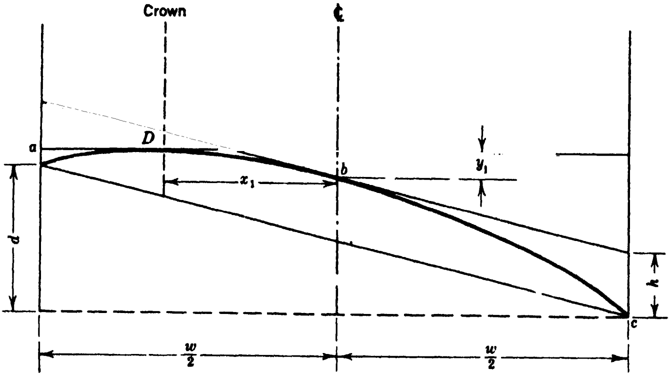


Fig. 31 Diagram illustrating method of designing an unsymmetrical crown.

Unsymmetrical Crowns. It is desirable to design street cross-sections with both gutters at the same elevation, and most cross-sections are so designed. On some sidehill streets, however, a symmetrical crown may result in excessive excavation or embankment with expensive retaining walls, and the appearance of the street may not be pleasing. If the unsymmetrical crowns occasionally used on such streets are carefully designed, they tend to create the optical illusion that the pavement crown is symmetrical. This practice ordinarily is confined to residential streets. The crown line or highest point of the cross-section is shifted to the high side of the street as shown in Fig. 31. Gutter depths are equal if possible, but sometimes it may be desirable to have a deeper gutter on the high side of the street than on the low side. The upper gutter should rarely have a depth greater than 9 in., and the lower gutter less than 4 in. Double-stepped curbs are generally used in place of high single curbs when excessive heights are involved. A single parabolic arc should be used unless some unusual condition requires a double arc. Referring to Fig. 31, the location and elevation of the crown line may be determined as follows:

Let w = width of pavement, in feet.
d = the difference in elevation of the gutters, in feet.
x_1 = the distance from the center line to the crown line, in feet.
y_1 = elevation of the crown line above the center line, in feet.
h = the amount of crown, in feet, for a street of width w feet.

The amount of crown, h, is determined as for a street with a symmetrical cross-section. The parabolic arc passes through the points a, b, and c, and has its vertex at some point, D. It may be shown that

$$x_1 = \frac{dw}{8h}$$

and

$$y_1 = \frac{d^2}{16h}$$

The elevation of any point on the parabolic arc may be determined by means of equation 12, substituting the crown line for the center line.

Horizontal Curves. Safety is the primary consideration in the design of horizontal curves, and the ideal curve therefore will be plainly visible for long distances, both in advance and along the curve, and will enable vehicle drivers to keep vehicles under control at reasonable speeds.

The length of road which is visible to a vehicle driver, or the *sight distance*, is intimately related to the stopping distances of motor vehicles at the normal maximum speeds in use. These stopping distances were discussed previously in this chapter. Minimum sight distances should provide some factor of safety, and 800 ft is considered desirable for primary and heavily traveled secondary state highways; 500 ft is a desirable minimum for primary and secondary local highways.

The sight distance along a horizontal curve is determined by the radius of the curve of the center line of the roadway and the distance from the center line of the roadway to any obstruction to view; it may be expressed by the formula

$$m = \frac{0.125S^2}{R} \tag{13}$$

in which m = distance from the center line of the roadway to the obstruction, in feet. (See Fig. 32.)

S = the sight distance along the center line of the roadway, in feet.
R = the radius of the curve of the center line of the roadway, in feet.

The assumption, in formula 13, that the arc distance ACB in Fig. 32 is equal to the sum of the chord lengths AC and CB introduces an error which is generally small and may be neglected.

The relation of headlamp sight distance to horizontal curvature also is important from the viewpoint of preventing collisions at night, particularly on high-speed highways. Figure 33 shows this relation as developed by the General Electric Company and the Pennsylvania Turnpike Commission.

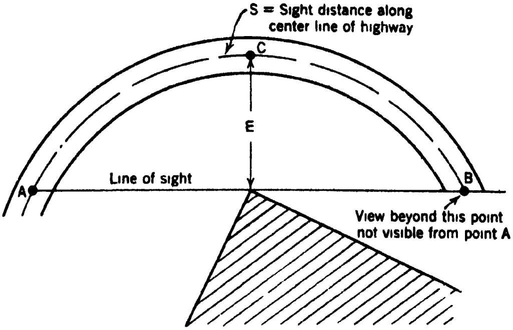

S = Sight distance along center line of highway
C
E
Line of sight
A
B
View beyond this point not visible from point A

FIG 32 Sight distance along a roadway on a horizontal curve.

Superelevation of Curves. The minimum desirable radius of curvature is intimately related to the extent to which curves may be safely superelevated to counteract the centrifugal force of vehicles traveling on the curve. Complete superelevation to balance the centrifugal force of vehicles, disregarding the effect of friction between the tires and the roadway surface, may be determined by the formula

$$e = \frac{V^2}{15R} \tag{14}$$

in which e = the superelevation, in feet per foot of width.
V = the speed of the vehicle, in miles per hour.
R = the radius of the curve, in feet.

If the friction between the tires and the roadway surface is taken into account, formula 14 becomes

$$e = \frac{V^2}{15R} - f \tag{14a}$$

in which f = the safe coefficient of friction, or safe side friction factor, for a speed V, in feet per foot.

The safe maximum rate of superelevation in areas subjected to snowfall is about 1¼ in. per ft or 0.10 ft per ft. The maximum side friction factor that can be developed before skidding occurs, according to R. A. Moyer, of Iowa State College, is about 0.30. Motor vehicles traveling at speeds of 50 to 60 mi per hr, however, may be difficult to keep in control with a factor as high as 0.30, and a factor of 0.16 has been recommended by the Project Committee on Relation of Curvature to Speed, of the Highway Research Board, for speeds up to and including 60 mi per hr with a reduction of 0.01 for each increase of speed of 5 mi per hr above 60 mi per hr. It is apparent, therefore, that the speed for which a curve is designed, and the selection of the side friction factor, will determine the minimum radius of curvature on a highway. If friction is disregarded, the minimum radius of curvature will be

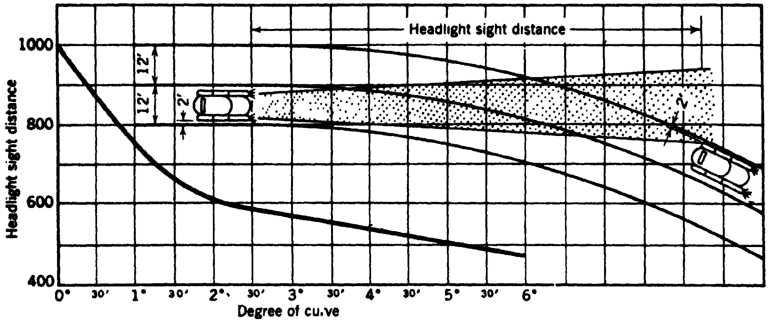


FIG. 33. Headlamp sight distances on horizontal curves as developed by the General Electric Company and the Pennsylvania Turnpike Commission. (*Charles M. Noble.*)

$$R = \frac{V^2}{15e} = \frac{V^2}{15 \times 0.10} = \frac{V^2}{1.5}$$

and if a friction factor of 0.16 is used, the minimum radius of curvature will be

$$R = \frac{V^2}{15(e+f)} = \frac{V^2}{15 \times 0.26} = \frac{V^2}{3.9}$$

The range in minimum radii under these two conditions is shown in Fig. 34. Stonex and Noble concluded from tests on the Pennsylvania Turnpike that a side friction factor of 0.10 is the maximum value which should be employed for speeds of 70 mi per hr and higher.

Prior to 1936, most highway organizations provided full superelevation (that is, disregarding side friction) for some assumed speed up to a maximum rate of about 0.10 ft per ft. A maximum speed of 60 mi per hr was quite widely assumed, and frequently, curves of less than 1° were not superelevated. Curves of 0° 15′ and 0° 30′ on primary roads now are often superelevated. The selection of a definite speed for the design of curves is essential in determining minimum radii and rates of superelevation, but no definite standards have been universally adopted. The selection of friction factors also is important, particularly in determining the minimum radius or degree of curvature at which superelevation will begin. Assuming a side friction factor of 0.16, an average crown of 0.01 ft per ft, and a speed of 60 mi per hr, the negative superelevation on the outer lane of a curve would be counteracted by a curvature calculated as follows:

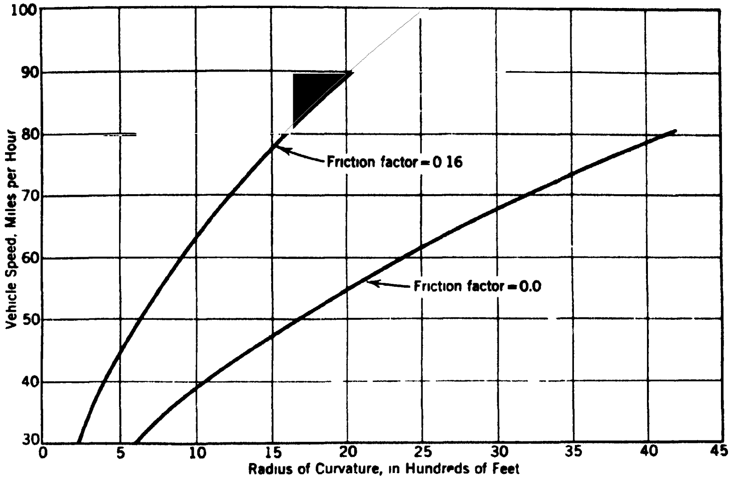


FIG. 34. Minimum radii of horizontal curves for various speeds; roadway superelevated at rate of 0.10 ft per ft of width.

$$R = \frac{V^2}{15(e+f)} = \frac{3600}{15(-0.01+0.16)} = 1600 \text{ ft}$$

This corresponds, approximately, to a 3° 30′ curve.

The adoption of standards relative to minimum radius and superelevation of curves for high speeds may necessitate large expenditures for additional right-of-way which cannot be justified on many highways, and varying standards will be necessary on the various classes of highways. The protection of traffic by means of effective lighting and signs will undoubtedly become essential on through-highways at locations where desirable minimum conditions of design cannot be realized.

Easement of Curves. Since no superelevation is required on tangents and full superelevation is required upon entering a circular curve, it is necessary to provide a gradual transition from the normal tangent section to the full circular curve section. This transition may be accomplished by means of transition or easement curves which may consist of compound circular curves, spiral transition curves (similar to those on railroads), or parabolic curves. The spiral curve is widely used, but for some reason the parabolic curve, which provides its own easement, has not been used in the United States. The crown of the roadway surface is gradually changed to a flat section on the approach to the easement curve, and the full superelevation is reached when the full circular curve is entered.

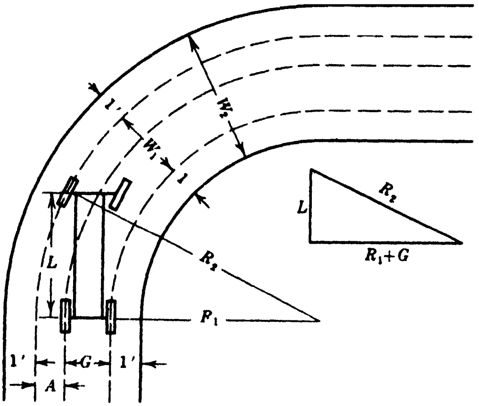


FIG 35 Extra width required in traveling round a highway curve

Widening on Curves. Owing to the fact that the rear wheels of motor vehicles do not ordinarily travel on the same radius on curves as the front wheels, it is desirable to widen traffic lanes on sharp curves. The theoretical amount of widening required is illustrated in Fig. 35 and may be calculated as follows:

$$R_2^2 = (R_1 + G)^2 + L^2 \qquad W_1 = R_2 - R_1 \qquad A = R_2 - (R_1 + G) \qquad (15)$$

in which R_2 = the radius of the inner rear wheel curved track, in feet.
R_1 = the radius of the outer front wheel curved track, in feet.
G = the gauge of the wheels, in feet (4 ft 8 in. for ordinary passenger cars).
L = the wheel-base length, in feet.

W_1 = the full width, in feet, required for clearance.
A = amount of widening required, in feet.

In addition to the amount of widening required for turning, it is desirable to provide a small additional width for psychological reasons. The widening generally is placed on the inside of easement curves in the shape of a lune. Present practice is to widen curves of 5° to 6° or more, the maximum widening being determined by means of the Voshell formula, which is

$$W = 2(R - \sqrt{R^2 - L^2}) + \frac{35}{\sqrt{R}} \tag{16}$$

in which W = the widening, in feet.
R = the radius of curvature, in feet.
L = the wheel base of vehicles, in feet (20 ft is recommended).

GRADES

Maximum Grades. The increase in gasoline consumption of motor vehicles is the principal item in the increased cost of vehicle operation on grades as compared with the cost of operation or level highways, and it will be shown in Chapter XVIII that grades up to 7½ per cent do not affect the cost of operation of ordinary passenger vehicles, provided that the rise and fall are equal along a highway. This maximum grade decreases as the weight of the vehicles and loads increases and may be as low as 2 per cent for heavy trucks loaded to capacity. Decreases in the cost of vehicle operation by grade reduction must be weighed against the cost of construction. An accurate solution of the problem is a practical impossibility because of the variations from year to year in the proportions of each class of vehicles which comprise the total traffic, and because of the constant changes in the design of motor vehicles. Therefore, the practical solution of the problem of grade establishment will depend upon the judgment of the engineer or engineers in charge of the design of a specific highway project. Maximum grades of 5 per cent in rolling country and 7 per cent in rough country are generally favored, but variations from these rates are often compatible with good judgment. Greater gradients for short distances are entirely feasible, particularly if they result in the elimination of excessive curvature.

Minimum Grades. Minimum grades on both rural highways and urban streets are governed by the necessity of provisions for drainage. Long fills on rural highways may be constructed with level grades, and level grades can be used on sections of either rural highways or urban streets where ditches or pipe drains may be constructed with sufficient grades to carry off water. Pipe drains should have inlets short distances apart under such conditions, and gutters must be sloped slightly or placed with great care to avoid "pockets."

Vertical Curves. Considerations of comfort and safety require that intersecting grades having differences in rate of more than 0.5 per cent be modified by means of vertical curves. The minimum length of vertical curves at summits will be governed by the distance within which two vehicles are within sight of each other, this distance being defined as the "sight distance." Proper sight distance varies for different conditions and should be greatest on roads having smooth pavements with no parking strips between opposing lanes of traffic. Recommended sight distances vary from 350 to 1000 ft for rural highways, the maximum being applicable to high-speed through-highways. Parabolic curves are used almost exclusively because of the convenience of calculating their coordinates. The length of parabolic curves for various minimum sight distances on crests under daytime conditions of visibility may be calculated by means of the formula

$$L = \frac{S^2G}{h} \tag{17}$$

in which L = the minimum length of the curve, in 100-ft stations.
S = the minimum sight distance, in 100-ft stations.
G = the algebraic difference between the two intersecting grades, in per cent.
h = the vertical height of the driver's eyes above the road surface, in feet (5 ft is generally used).

Lengths of vertical curves on rural highways are generally multiples of 50 ft, and odd lengths obtained by means of the preceding formula are generally increased to the next largest multiple of 50 ft.

Sight distance at night is an important consideration on high-speed through-highways to prevent collisions with unlighted objects. Figures 36 and 37 show the relations of headlamp sight distances developed by the General Electric Company and the Pennsylvania Turnpike Commission.

The calculation of ordinates to parabolic vertical curves is illustrated in Fig. 38. The point d is the mean in elevation of points c and t, the two ends of the curve. The point e is half way between the points a and d, and may be determined by direct calculation as described or by means of the formula:

$$H = \frac{G_1 - G_2}{8} \times \frac{L}{100} \tag{18}$$

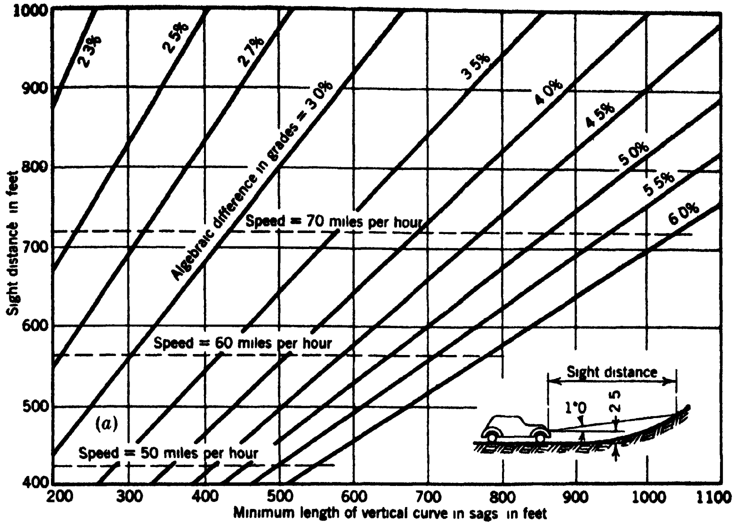


Fig 36 Headlamp sight distances on vertical curves on sags as developed by the General Electric Company and the Pennsylvania Turnpike Commission (*Charles M Noble*)

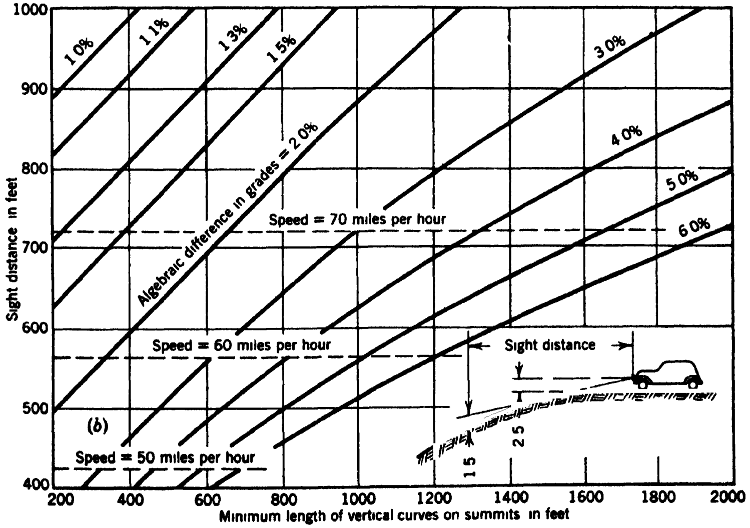


Fig 37 Headlamp sight distances on vertical curves on crests as developed by the General Electric Company and the Pennsylvania Turnpike Commission (*Charles M Noble*)

in which H = the vertical distance, in feet, from the intersection of the grade tangents to the midpoint of the curve.

$G_1 - G_2$ = the algebraic difference, in per cent, between the two intersecting grades, always expressed as a positive number.

L = *the length of the curve, in feet.*

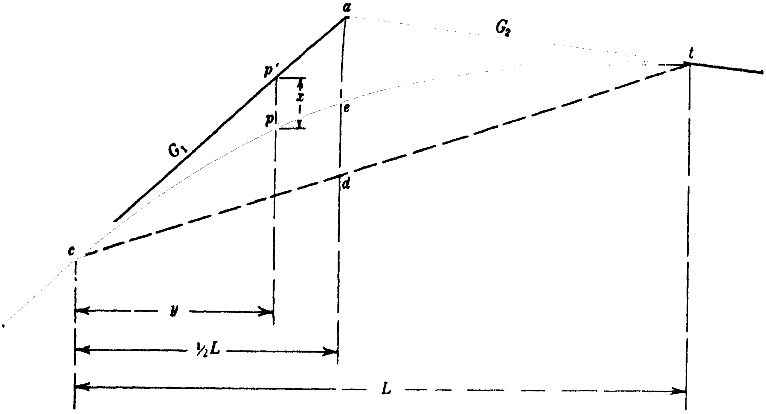


FIG. 38. Diagram illustrating method of computing the ordinates of a vertical curve

The elevation of any point on the curve, such as p, may be determined by first computing the elevation of the corresponding point p' on the grade tangent, and then computing the difference in elevation between the points p and p' by means of the formula:

$$x = \frac{y^2H}{\left(\frac{L}{2}\right)^2} \tag{19}$$

in which x the difference in elevation between points p and p'.

y the horizontal distance of point p from the nearest end of the curve.

H and L have the same significance as in equation 18.

It will be observed that the foregoing method introduces a slight error when c and t are not of the same elevation, but it is so small that it may be neglected.

The number of points of which the elevations should be computed for earthwork calculations will depend upon the difference in rate of the intersecting grades and upon the length of the vertical curve. It should be such that the points may be connected by straight lines, the difference in grade of which does not exceed 0.4 per cent. The distance between successive points thus computed will vary from 10 to 50 ft, the intervals usually adopted being 10, 20, or 50 ft. The number of points computed may also be determined by the distance between grade stakes, *although this degree of refinement is unnecessary except where the difference between intersecting grades is large and the vertical curve is relatively short.*

It is not possible under all conditions of topography, even on important heavily traveled roads, to obtain desirable minimum sight distances, and some highway departments have adopted the practice of providing an additional traffic lane on each side of the roadway, on such locations, to decrease the accident hazard.

Street Grades. The establishment of street grades often is a much more complicated problem than on rural highways because of the influence of adjacent property and the closer spacing of intersections. Streets differ from rural highways in one very important respect—rural highways generally are designed so that as little surface water as possible is carried along the highways, whereas municipal streets are generally designed to receive all surface water from adjoining property and carry it to openings into underground drains. Street grades therefore are generally in cut, and no attempt is made on most street improvements to balance the earthwork in cuts and fills. Appearance and connections with abutting property are other important considerations in the establishment of street grades.

A minimum grade for streets of 0.25 per cent is desirable in order that surface water may be quickly carried off by the gutters. Vertical curves are used on street grades at abrupt changes in grade to facilitate comfortable riding and to create a good appearance. Sight distance is not so important as on rural highways, except in unusual instances, and the appearance of the street is the most important factor in determining the lengths of vertical curves; a frequently used rule in cities is to make the length of vertical curves, in feet, not less than twenty times the algebraic difference between the intersecting grades.

HIGHWAY INTERSECTIONS

Conditions Affecting Design. The multitude of conditions which affect the design and treatment of highway intersections include the class and type of each intersecting road, the volume and nature of the traffic on each road, the topography, the number and direction of the intersecting roads, the amount of turning movement, and the development of the land adjacent to the intersection.

Types of Intersection Treatment. The treatment of intersections in the improvement of highways may be classified as follows:

1. General features common to most intersections.
2. Enlarged intersections.
3. Separation of the highway grades.

General Features of Design. General features of design which are common to almost all intersections consist of restrictions relative to the alignment and grades of the approaches, the grades across the intersection, the installation of warning signs, and the elimination of obstacles to visibility.

The grades across intersections should be level or as nearly level as possible, and approach grades should not exceed 1 per cent for distances which depend upon the class and type of the highway; 75 to 100 ft is a desirable minimum distance for approaches on purely local or land-service roads, whereas 300 ft is desirable on through-highways. Similarly, proximity to curves on which the visibility is limited should be avoided.

The installation of warning signs deserves careful thought. Such installations easily can be overdone and thus lose a large measure of their effectiveness. It is unnecessary, on through-highways, to install "Cross Road" or "Side Road" signs in advance of all intersecting local or land-service roads; appropriate signs should, however, be placed on the local roads. Advance warning signs should be installed on through-roads on approaches to intersections with other through-roads.

Natural obstacles should be eliminated if possible. Slopes in cuts adjacent to intersections may be flattened, and hedges and similar obstructions to visibility can be removed. These obstacles cannot always be removed on approaches on local roads but deserve careful attention on through-roads. Billboards sometimes create hazards at intersections, and some states have enacted laws prohibiting billboards within specified distances from intersections.

Enlarged Intersections. Four types of enlarged right-angle intersections are shown in Fig. 39. The paved intersection is widely used at the intersection of two paved roads, each of which continues straight through the intersection (for example, the intersection of a secondary and a primary county highway) and may consist merely of rounding off the corners at the edges of the pavements without acquiring additional right-of-way. Additional right-of-way frequently is obtained at paved intersections to increase visibility and to provide for probable future changes in the intersection layout.

The curved type of intersection is common at the intersection of two heavily traveled roads, such as two primary state highways; the radii of the curved portions should not be less than 300 ft, and sometimes they are much greater. The intersection is widened at locations where the curved intersection is impracticable on account of cost or topographic conditions. The traffic circle is constructed to move traffic in one direction around a circle or similarly shaped intersection; it includes a central island, which forces traffic to take a rotary course, and generally directional islands which cause the traffic to enter the intersection through definite channels and move into the rotary traffic lanes at oblique angles. Traffic circles are particularly well adapted to multiple intersections in cities but also are used on rural roads.

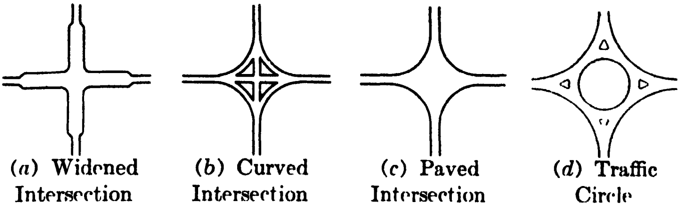

(*a*) Widened Intersection (*b*) Curved Intersection (*c*) Paved Intersection (*d*) Traffic Circle

FIG. 39. Types of enlarged highway intersections

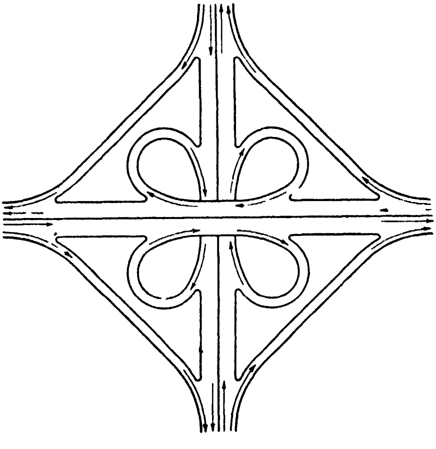


FIG. 40. "Cloverleaf" highway grade separation.

Highway Grade Separations. The separation of highway grades, except under very favorable topographic conditions, is a costly procedure, and grade separation has been undertaken only on highways subject to very large volumes of traffic.

Deck structures are commonly used, being well adapted to highway grade separations on account of their appearance and lack of interference with visibility. Sometimes the lower roadway is depressed to fit the location.

The "cloverleaf" pattern of grade separation layout, in which all turning traffic turns to the right, is desirable when turns must be provided at intersections, because traffic lines do not cross and through traffic need not be interrupted. Numerous modifications of the standard pattern have been adapted to meet special conditions.

Separated Crossings for Pedestrians. The use of separated crossings for pedestrian traffic is justifiable on wide through-highways in congested areas, particularly near schools. The subway is the usual form of separated pedestrian crossing.

STREET INTERSECTIONS

Street Intersection Grades. The treatment of grades at street intersections is sometimes a perplexing problem, and many methods have been devised for handling this feature of design. The most troublesome intersections are on streets having grades in excess of 2 to 3 per cent. The treatment of grades may embrace carrying the street grade across the intersection, or flattening the intersection area by breaking the street grades at the curb lines or property lines. A method which was developed by V. S Moon, while Assistant Engineer of the Board of Estimate and Apportionment of New York City, is given in Appendix A.

Curb Radii. The radius of curbs at intersections is intimately related to traffic convenience and safety Short-radius curves of 4- to 8-ft radius were the general rule in municipal street design for many years, but longer radii are now used. Theoretically, the minimum radius of curves at intersections should be equal to the turning radius of a streetcar on streets having car tracks and to the longest turning radius of all but exceptional motor vehicles on streets without car tracks; 50- and 25-ft radii, respectively, would be required to fulfill these requirements. General practice limits curb radii to 15 to 30 ft.

Widening Intersections. Splaying or widening of street intersections sometimes is practicable, particularly on streets having car tracks, to facilitate the movement of traffic. The volume of traffic, the volume of turning traffic, and the availability of right-of-way are important considerations in connection with widening. Since widening is necessary on only one side of a street at the entrance to an intersection, the use of the right lane adjacent to the curb generally should be restricted to turning traffic.

Rotary Intersections. Rotary intersections generally are circular, or nearly circular, in shape, although elongated ovals are used in some locations; they are particularly adaptable to multiple intersections. Since they require large areas of ground, they usually must be incorporated in the original city plan.

Provisions for Safety of Pedestrians. Numerous provisions for the safety of pedestrians at street intersections are employed. Painted stripes outlining the limits of pedestrian crossings are common. On streets paved with concrete, pedestrian crossings constructed with white Portland cement are utilized to some extent, the adjacent normal concrete being slightly darkened for contrast by the addition of a small proportion of carbon black in the paving mixture. The construction of separated pedestrian crossways, usually in the form of subways, is desirable at unusually heavy traffic crossings, and is increasing in the larger American cities.

RAILROAD GRADE CROSSINGS AND GRADE SEPARATIONS

The design of highway and railroad crossings at grade is an individual problem in each case, and the details of design depend upon the angle of crossing, the railway and highway grades, the visibility, the railway and highway alignment for considerable distances from the crossing, the volume of traffic on both the railroad and the highway, drainage, and other conditions The following conditions represent an ideal highway-railway grade crossing:

1. A 90° angle of crossing.
2. The crossing should be visible for 500 ft on either side of the railroad.
3. The railway should be visible for 1000 ft on either side of the highway at a distance, on the highway, of 300 ft from the crossing.
4. The railway alignment should be straight.
5. The highway grade should be level for a distance of 100 ft on each side of the crossing.
6. The grades of the highway beyond the level section should not exceed 3 per cent for distances of 500 ft.
7. Sharp curves and highway intersections should not be less than 250 to 750 ft from the crossing, the distance depending upon the importance of the highway.
8. The paving in the railroad area should be smooth and of the same width as the highway but not less than 20 ft. Single-lane or narrow two-lane highways should be widened to 20 ft for at least 200 ft on each side of the crossing.

9. **Warning signs should be installed not less than 500 to 750 ft from the crossing.**
10. Warning signals, consisting of flashing lights, should be installed on each side of the crossing.
11. Crossings should be lighted at night, when necessary, and the lights should be so arranged that passing trains will be plainly visible.

Obviously, not all these ideal conditions can exist at all grade crossings, and modifications are necessary on the more lightly traveled roads; when ideal conditions are not obtainable on heavy-traffic highways, grade separation is desirable.

Paved Railroad Crossings. Pavements for railroad crossings at grade are of various types.

Plank Crossings. Plank crossings are widely used for paving crossings on light-traffic roads and are satisfactory when adequately maintained White oak is one of the most satisfactory woods, but untreated and creosoted southern pine and fir are also used.

Bituminous Crossings. Bituminous materials of all kinds are used for paving grade crossings. Rock asphalts and cold artificial mixtures are favored because of their convenience in handling and ease of maintenance.

Concrete Crossings. Cast-in-place concrete crossings have been used quite extensively but are not entirely successful on account of the destructive effect of vibration from trains.

Precast Concrete Slabs. Precast concrete slabs possess the advantage of ease of maintenance They must be heavily reinforced and should be provided with connections for handling with truck cranes.

Rail Crossings. Crossings constructed with steel rails are widely used and are very satisfactory except on skew crossings at sharp angles. Old steel rails are utilized, which require the placing of leveling strips if the rails are smaller than the track rails.

Traffic Treads. Steel traffic plates make very satisfactory paving for grade crossings. They necessitate sleepers and planks on the ties for support.

Separation of Grades. The separation of railway and highway grades is essential in many locations to eliminate dangerous crossings; on heavily traveled highways, it is desirable on account of the savings in operating costs of vehicles.

Grade-Separation Structures. Grade-separation structures may consist of an overpass, which carries the highway over the railroad, or an underpass or subway, which carries the highway under the railroad. The selection of either class of structure depends upon the topographical conditions at each crossing.

Girder bridges are used extensively for grade-separation structures on account of their adaptability, cost, and appearance. High steel trusses also are used where span lengths make girders impracticable. Many rigid-frame structures, both of steel and of concrete, have been used.

The alignment of approaches to grade crossing structures should be such that sight distances of 500 to 750 ft are obtained. Approach grades should never exceed 5 per cent. Underpasses should not be obstructed by center supports for the overhead railroad structure unless the traffic lanes are divided by center parking strips and protected by curbs.

The standard railroad clearance for the highway overpasses is 22 ft, and the minimum clearance should be 14 ft for highway underpasses except when electric railway tracks are present, in which case a minimum clearance of 16 ft is desirable. In some locations where subways are desirable, it is necessary to depress the roadway for considerable depths below the surrounding ground elevation, and drainage becomes a problem; automatic sump pumps are used under such conditions to remove surface water from rains. Failure of electric power or machinery creates great inconvenience or even a dangerous hazard, and therefore this design should be avoided if possible.

Laws Relating to Railroad Crossings and Grade Separations. All states have laws which outline the procedures necessary to improve highway and railway intersections, and most states have enacted laws requiring definite warning and control devices for grade crossings. Michigan, for example, requires the use of reflectorized crossing signs at all grade crossings not protected by flashing-type signals. The participation of railroads in the cost of grade separations usually is controlled by public utility commissions.

EXPRESS HIGHWAYS

An express highway is a road or street designed for large volumes of traffic operating at controlled moderate to high speeds for considerable distances with a minimum amount of interference or interruption from cross-traffic or entering and leaving traffic. Thus an express highway may be considered a special type or class of arterial or through-traffic street or road. The distinguishing feature of a fully controlled express highway is the incorporation in the design of a freeway, which has been defined as a roadway "to which there is no vehicular access from abutting property." Several express highways have been built in large cities, and non-interference with traffic has been secured by the elevation, partial elevation, or depression of the roadway and the construction of grade separations for vehicular traffic and pedestrians. A few rural roads with long freeways have been built in metropolitan areas.

Express highways have been suggested for interstate and primary state highways with divided roadways, no highway or railway crossings at grade, separate service roads for adjacent property, separated pedestrian crossings, long sight distances, long and fully superelevated curves, provisions for the safe removal of disabled vehicles, and fully controlled minimum and maximum speeds. Roadways and other features designed for speeds of 100 mi per hr have been suggested for such highways. This type of construction would involve very large expenditures, of course, and it presents many difficult problems. Infrequent crossings would seriously interfere with access to rural property, and the control of speed limits would be expensive. The necessity of competing with air travel by means of high-speed, expensive land roadways deserves serious consideration. Many of the designed features of express highways, however, may be, and are, incorporated in plans for the improvement of many primary highways.

MISCELLANEOUS HIGHWAY STRUCTURES

Guard Railings. The various types of guard rails on highways include wooden guard rails, cable and wire tape rails, wire mesh rails, steel plate railings, and masonry railings.

Wooden railings consist of posts and rails Lumber of small cross-sectional area has low resistance to shock and may cause great damage to vehicles and occupants. Heavy planks, 3 by 10 in. and larger, firmly fastened to square, creosoted timber posts are quite common in some sections of the country and represent good practice in the use of timber.

Steel cable railings usually consist of ¾-in wire rope mounted on wood posts; steel posts and reinforced-concrete posts also are used. Round cedar posts are favored because of their durability; the lower ends of the posts preferably should be treated with creosote oil or creosote-coal tar solution by dipping or brushing. Steel cable railing is low in cost of installation in comparison with other railings but has a number of disadvantages, including low visibility of the cable, sagging due to changes in temperature, and incomplete protection to vehicles when struck. Spring tightening devices and offset lugs to hold the cables away from the posts decrease the last two disadvantages to a considerable extent, and a durable white paint on the posts provides well for visibility. Wire tape rails are used in place of cable rails, but their additional value is perhaps questionable.

Wire mesh guard rails are from 14 to 22 in. wide and are usually placed on wooden posts. They generally are somewhat more expensive than cable railings but offer less tendency toward damage to vehicles when struck; they are strong and tend to deflect vehicles back onto the roadway. Their visibility is generally good. The principal objections to this type of guard rail are the necessity of complete replacement of sections which are struck and the fact that, in snow regions, the railing acts as a snow fence.

Numerous types of steel-plate guard rails are used, some of which are attached directly to the posts while others have spring connections which permit the railings to deflect when struck. These railings are very effective in preventing damage to vehicles; they are visible when properly painted and are easily installed; their disadvantages are their cost and their tendency to act as snow fences in regions subject to snowfall.

Masonry railings are used occasionally under special conditions on account of their appearance. They are costly to install.

The matter of guard rail on individual highways and on highway systems as a whole deserves very careful study. Guard rails are placed, of course, on embankments, along deep ditches, and along natural watercourses. Guard rails are expensive to construct and maintain; they interfere with roadside mowing, shoulder maintenance, and snow removal; and often they constitute hazards in themselves. Large numbers of county highway systems contain large amounts of guard rail which are inadequately maintained Guard rail has a definite place in road design, but its use has been greatly overdone in many areas. Numerous local highway organizations have studied the guard-rail installations on their highway systems and have outlined carefully prepared plans for their elimination Such plans, under some financing conditions, may require many years for completion, but at least they represent intelligent efforts to deal with a perplexing problem.

Signs. Highway signs are an essential part of any highway system, and their design and installation have become quite well standardized through the efforts of the American Association of State Highway Officials. Highway signs may be classified into several groups, consisting of road markers, warning signs, caution signs, restriction signs, and direction signs.

Road markers identify routes of travel and are of great value on through-roads; their value on secondary county highways and on land-service roads is questionable. "Right" and "left" route markers, bearing the letters R or L, respectively, indicate the direction of highway routes at approaching road intersections. Standard route markers have black letters on a white background.

The standard railroad and "stop" signs, which are circular and octagonal, respectively, are well known and need no detailed description. Diamond-shaped signs indicate conditions of inherent danger such as curves, winding roads, and narrow bridges. Square signs indicate the necessity of caution because of slippery or loose road surfaces, soft shoulders, school zones, and other special conditions. All these signs are normally 24 inches in least dimension and have black letters on a yellow background.

Restriction signs serve to indicate legal restrictions such as speed limits, load limits, parking restrictions, and regulations regarding turns. These signs generally have black letters on a white background.

Direction signs giving mileage, state and county boundary lines, names of towns, streams, and other topographical features are desirable to facilitate through-traffic. Arrow-shaped direction signs are suitable for local roads and for indicating turns on through-roads; the square sign indicating "straight-ahead" routes and destinations combines marker and direction signs, and facilitates travel on state highways by being placed on approaches to intersections and facing approaching vehicles.

Most highway signs are made of pressed metal on account of cost and ease of handling. Both wood and steel posts are used. Reflector buttons are desirable to facilitate night driving, particularly on through-roads.

Highway Lighting. The complete lighting of highways is not practicable except on roads which have heavy night traffic, but partial lighting is desirable on many highways to overcome hazards due to physical features of design and construction which are not always easy or economical to eliminate. Two types of lighting units are most extensively used: the high-pressure mercury-vapor lamp combined with a tungsten-filament incandescent lamp, which provides a white light and is considered very desirable on urban thoroughfares; and the sodium-vapor lamp, which produces a monochromatic, yellow light and is considered very satisfactory for rural highways because of the silhouetting of objects.

Roadside Improvement of Highways. In the initial stages of highway improvement in this country the construction details naturally received the greatest attention. As construction programs began to keep pace with traffic developments, more attention was devoted to the improvement of roadsides.

The principal objectives of roadside improvement are the promotion of safety, the simplification of maintenance operations and reduction of the cost of maintenance, and the esthetic development of the highway and adjacent property. These objectives are interrelated, and frequently methods of accomplishing one will contribute to another. The general problem involves not only features of design but methods of construction and maintenance as well.

The promotion of highway safety by means of roadside improvements requires that careful attention be given to the selection of shoulder widths, the elimination of steep slopes on embankments, the elimination of deep ditches, the elimination of obstacles, the construction of sidewalks for pedestrians in some locations, and adequate provision for parking in areas where the lack of such facilities may be a serious menace to safety. Most of these features already have been described in this chapter. Although maintenance operations may not, in a narrow sense, be construed as features of roadside improvement, nevertheless such items of work as mowing and snow removal are an essential part of the general program of promoting the highway safety and should be carefully scheduled to eliminate and prevent the presence of traffic hazards.

Careful attention in the design and construction of highways will eliminate many costly items of maintenance. Adequate financing of roadside improvements often presents a difficult problem, particularly in connection with local road improvements, and, in many areas, a carefully planned long-range program must be adopted to achieve adequate results. Very frequently the difficulty of providing for proper roadside improvements of local roads, such as primary and secondary county roads and land-service roads, is not fully appreciated.

The operations of mowing and snow removal are seriously affected by roadside conditions. Narrow shoulders, deep ditches, rough contours, the presence of large boulders and sometimes of trees and pole lines may reduce the amount of machine mowing and make it necessary to resort to more costly hand methods. This problem often is acute on local road systems in rolling or hilly country. Similar conditions often increase the cost of snow removal, especially when repeated snowfalls occur at short intervals; if the shoulders are narrow and the roadsides are rough, the snow cannot be moved outward to provide room for removing later snowfalls.

In rolling or hilly country, the adequate maintenance of guard railing often is a significant item in maintenance budgets, particularly on local roads. Not only is their presence an obstacle to mowing and snow-removal operations but also their upkeep (except in the more costly types) requires frequent replacements, painting, and tightening. Guard railing should be eliminated during construction wherever possible by constructing flat side slopes and eliminating deep side ditches.

One of the most important features of roadside improvement, from the viewpoint of maintenance cost as well as that of roadside appearance, is the adoption of effective measures for controlling erosion. The shoulders of highways are subjected to erosion from roadway water, side slopes are eroded by roadway water and water from adjacent land, and ditches and gutters often are eroded by runoff water. The following methods of controlling erosion are commonly employed:

1. The selection of suitable cross-sections.
2. Installation of appropriate provisions for cross-drainage to reduce the volume of water along the highway.
3. The construction of lip curbs or separate paved gutters immediately adjacent to pavements on soils which erode easily.
4. The stabilization of shoulders by oiling, graveling, and other methods.
5. Paved gutters and ditches.
6. The installation of pipe drains for carrying storm water (as distinguished from pipe drains for carrying only the seepage of ground water in comparatively small quantities).
7. Installation of ditch checks in erosive soil.
8. Seeding, sodding, mulching, and oiling of slopes.
9. The rounding of slopes, particularly on side cuts and on embankments.
10. The construction of intercepting ditches on side slopes on adjacent land.
11. The installation of drop inlets at the upper ends of culverts.
12. The suitable design of culverts with particular attention to headwalls, cutoffs, and aprons.
13. The control of slides.
14. The planting of trees and shrubs.

Most of these features have been described in this or a preceding chapter or will later be described at appropriate places in the text. Attention is here called to the nature of control measures as a fundamental part of roadside improvement.

The esthetic improvement of highways is concerned with the development of a roadside which blends into and harmonizes with the surrounding landscape It is impracticable, within the limitations of a book of this sort, to devote a large amount of space to a separate treatment of this important phase of highway improvement. For our present purpose, attention will be called to the principal features of esthetic development These are:

1 The harmonizing of construction with the topography For example, slopes should be rounded, tree and shrub plantings should consist of native growths arranged naturally, and native stone should be utilized for ornamental structures
2 Rights-of-way of existing roads should be cleaned of debris and unattractive trees and other vegetation
3 The utilization and development of existing scenic attractions
4 The control of billboards and pole lines
5 The planting of native trees and shrubbery at strategic locations
6 The care of trees and shrubbery
7 Erosion control
8 The development of parking areas, turnouts and overlooks for recreation areas and scenic attractions

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CHAPTER V

EARTHWORK

Scope. The earthwork, or grading operations, in connection with highway or street improvement, embraces excavation and the formation of embankments, and may include clearing and grubbing and other operations, such as seeding, sodding, moving of fences, and various roadside improvements.

Clearing and Grubbing. Clearing consists of the removal of trees, brush, logs, and debris; and grubbing consists of the removal of roots and stumps. Both these items of work usually are small in volume on rural road improvement but may be large on new locations.

GRADING MACHINERY

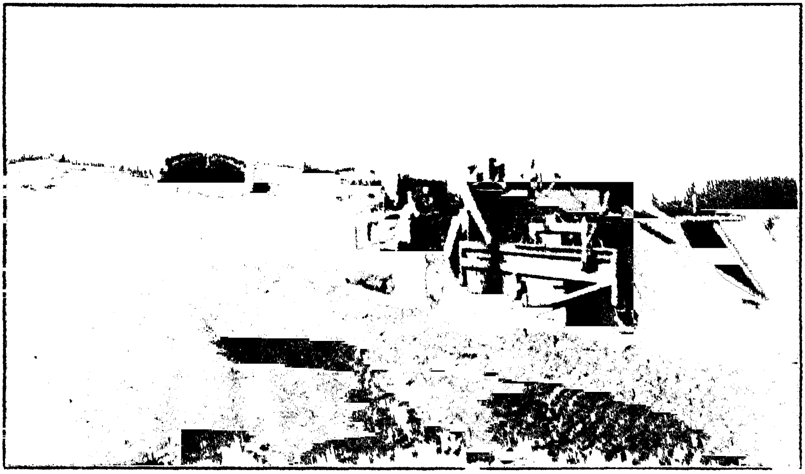
Loosening Equipment. Separate equipment sometimes is necessary for loosening earth prior to excavation, but many types of excavating machines require no loosening. Blade graders, elevating graders, and power shovels generally do their own loosening. Plows, scarifiers, and road rooters are used to loosen earth and old low-type wearing surfaces for scraper excavation. The ordinary mold-board plow and the railroad plow are suitable on small operations in connection with horse-drawn scraper excavation. Heavy graders equipped with heavy scarifier teeth are very satisfactory scarifiers. Road rooters consist of heavy, curved teeth mounted on a framework carried by wheels and are frequently used in connection with blade graders and scrapers.

Scrapers. Scrapers for excavation are of many types and may be classified roughly as horse-drawn and tractor-drawn.

Horse-Drawn Scrapers. Horse-drawn scrapers were widely used at one time but have become almost obsolete as the result of the development of crawler-type tractors. The slip, or drag, scraper, the Fresno scraper, and the two-wheel scraper were the most common scrapers and still are found to some extent on small operations.

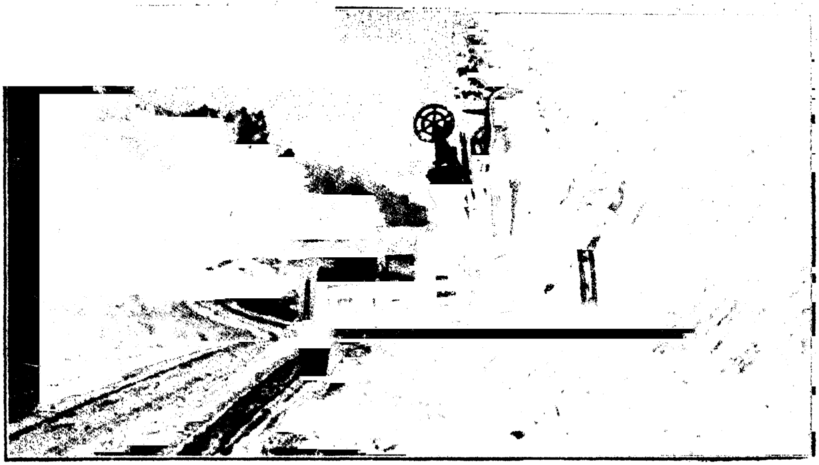
Tractor-Drawn Scrapers. A large number of tractor-drawn scrapers are available for excavation; they range in capacity from 1 to 12 cu yd. Some of these scrapers are drag scrapers, that is, they scoop up the earth and drag it over the ground; these are best adapted to grading operations in which the excavated earth is placed in nearby embankments. The type of scraper illustrated in Fig. 41 is built in several sizes up to 12-cu-yd capacity and is adaptable to long hauls up to 2000 ft or more.

Blade Graders. Blade graders are made with blades ranging in length from 7 to 14 ft and are used for both construction and maintenance operations. They are especially useful for forming grades where the amount of longitudinal movement of earth is small and where the subgrade or earth road surface and the ditches or gutters are built simultaneously by moving earth from the sides to the center of the roadway. They are effective in combination with scrapers and power shovels for loosening earth, making side slopes, forming shallow ditches or gutters, shaping shoulders, and forming and shaping subgrades and earth wearing surfaces. An 8-ft grader ordinarily requires a 35- to 40-hp crawler-type tractor; a 10-ft grader requires a 50- to 65-hp tractor; and 12- to 14-ft graders require 75- to 100-hp tractors. Graders used for forming grades should be of the leaning wheel, "high-lift" type. The high-lift arrangement permits swinging the blade outward beyond the outside wheel line to cut a bank slope. An independent scarifier (that is, mounted separately from the grader blade) is a desirable accessory for blade graders. Blade graders of larger size may be obtained with power units for operating the blades



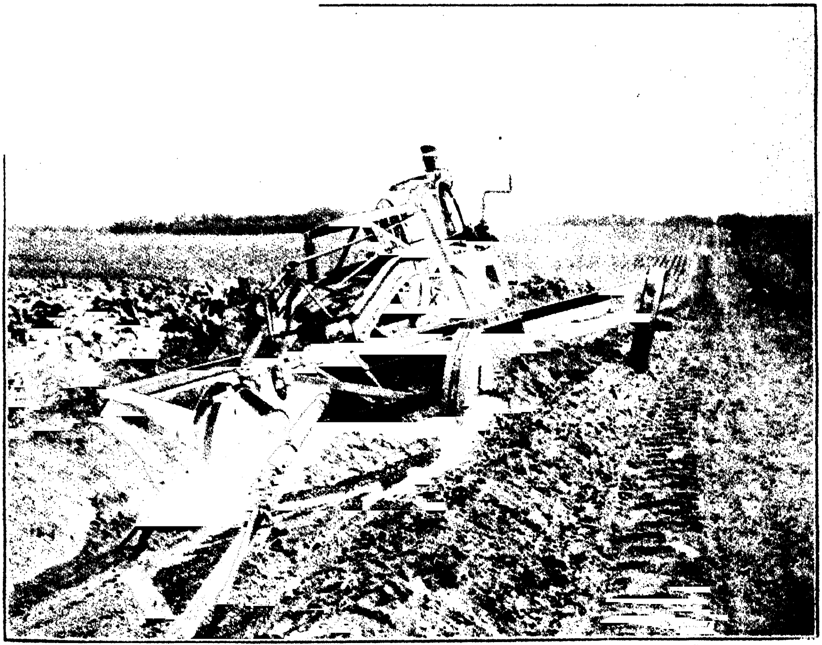
Courtesy of R G Le Tourneau, Inc

FIG 41 Twelve-yard tractor-drawn wheel scraper



Courtesy of J. D. Adams Co.

Fig. 42. Blade grader cutting outside bank slope.



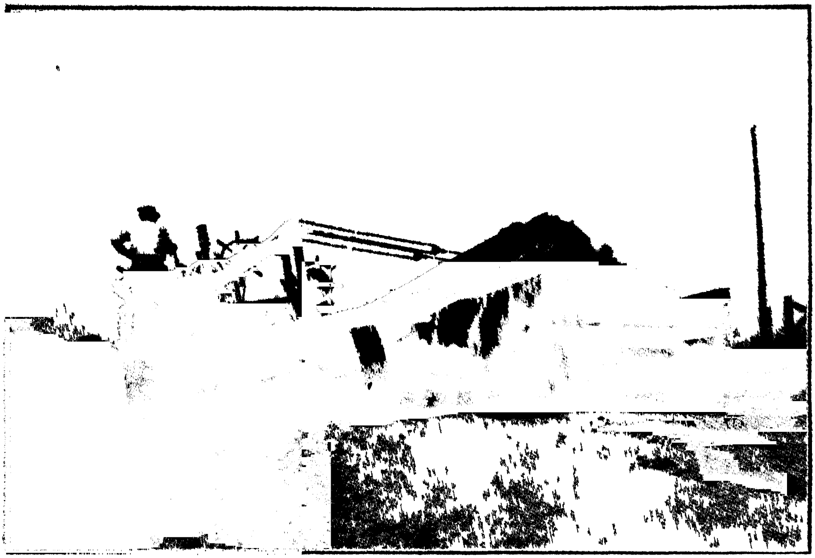
Courtesy of J. D. Adams Co.

Fig. 43. Cutting ditch with heavy blade grader. Note backsloping attachment.

which otherwise require a large amount of hard physical work on the part of the operator.

The self-propelled grader (see Fig. 80) is for forming subgrades and earth road surfaces under favorable conditions but is more widely used on maintenance operations than on construction operations.

Elevating Graders. The elevating grader consists of a steel frame mounted on two-wheel trucks, the rear wheels having a much greater diameter than the front wheels This steel frame carries a plow and a transverse, inclined endless belt. The inclination and the length of the belt are changeable Either a mold-board or a disc plow may be used for loosening the earth, which is then carried up on the inclined belt and dumped into wagons or deposited in banks parallel to the excavation. Elevating graders may be obtained in several sizes. One standard make weighs about 15,000 lb and has a 48-in. carrier with adjustable lengths from 19 to 25 ft The elevating grader has a rather restricted field of use on highway construction It is most efficient on nearly level ground, and soil conditions must be favorable; very light soils cannot be lifted onto the conveyor belt; wet, sticky soils are hard to excavate, and boulders and roots make it impossible to operate an elevating grader. It is useful for excavating wide shoulders and flat ditch slopes on semi-improved roads where the ex-

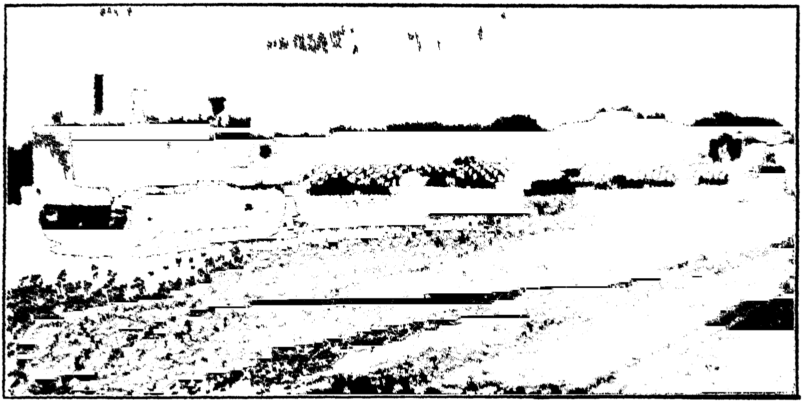


Courtesy of J D Adams Co

FIG 44 Elevating grader loading into truck

cavated soil must be moved for long distances in trucks. The standard size of elevating grader will excavate 40 to 80 cu yd per hr under average conditions.

Power Shovels. Gasoline and Diesel-driven shovels are widely used on highway and street grading operations and are operated in connection with motor trucks and various types of dump wagons or trailers hauled by motor trucks or tractors. Self-propelled shovels mounted on crawler treads are used almost exclusively, and their sizes are controlled by the convenience and cost of transporting them on trailers and by legal limitations on the size and weight of vehicles which may be hauled on highways and streets The dipper capacities of shovels therefore range from ⅜ to 1 cu yd; occasionally 1¼-cu-yd shovels are used, but they generally are too cumbersome to transport on trailers. Full revolving shovels are desirable for convenience in loading Truck-mounted shovels are employed to some extent as they possess the advantage of rapid mobility; they necessarily are restricted to small sizes and therefore are economical on small improvement projects; their greatest value is on maintenance operations and for loading of materials.



Courtesy of Blaw Knox Co

FIG 45 Compacting a road embankment with sheepsfoot rollers

The field of use of power shovels is large. They generally are most economical on projects involving large volumes of excavation and long hauls, but they also are efficient for handling long sections of shallow excavation where prior loosening is unnecessary. For this reason, in addition to their ability to work to very close limits, they are widely used for excavating old gravel, broken stone, and similar wearing surfaces on roads and streets. Since street pavements are generally depressed below the natural ground surface, the earthwork on streets consists almost entirely of excavation which must be hauled to some convenient location for disposal; power shovels, therefore, are ideal machines for excavating streets prior to paving. The capacities of shovels of the sizes mentioned vary from about 20 to 50 cu yd per hr, depending on their size, the nature of the excavation, and the amount of moving necessary.

Motor trucks are the most satisfactory hauling vehicles for shovel excavation if the road or ground surface over which they must travel is suitable for their operation. The most efficient size of truck depends upon the length of haul of the excavated material. Trucks having dump bodies of 2- to 3-cu-yd capacity are generally satisfactory for short hauls up to 2000 to 3000 ft. Truck trailers equipped with dump bodies are sometimes used under favorable hauling conditions. When hauling conditions for motor trucks are unfavorable and the character of the excavation is not adaptable to scraper operation, tractor-drawn wagons or buggies must be resorted to.

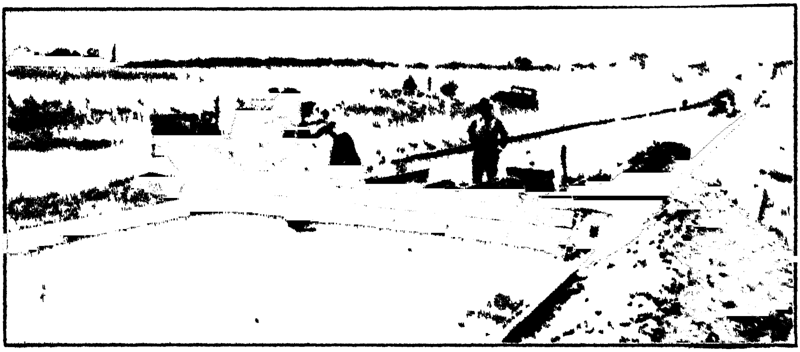
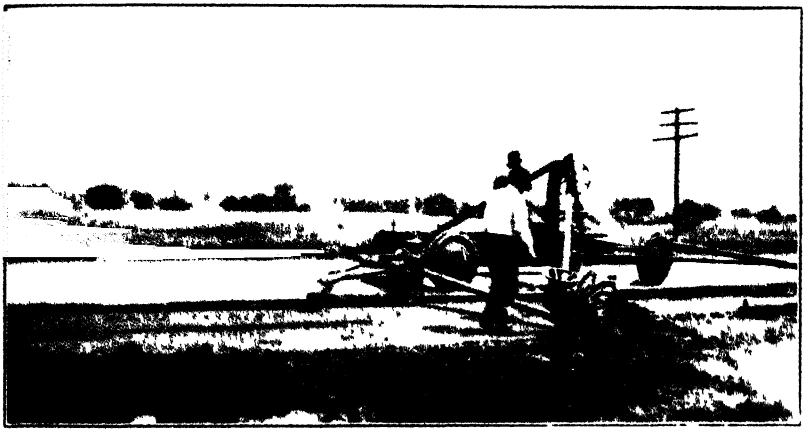

Courtesy of R B Equipment Co.

FIG. 46. Power-operated subgrading machine in use on a concrete paving project.

Cranes and Drag Lines. On highway grading projects, cranes are frequently equipped with clam shells or drag line buckets, and drag line cableways. The former are useful for some excavation and backfilling operations for structures and also for excavating and filling swamps and sinkholes.

Bulldozers. A bulldozer consists of a heavy steel blade which is mounted on a frame attached to a crawler-type tractor and is useful for backfilling and pushing earth over the sides and ends of embankments.

Rollers. Rollers are used in connection with grading operations for compacting subgrades under wearing courses, for compacting shoulders, and for consolidating embankments. The 10-ton, three-wheel roller is quite generally favored for rolling subgrades and shoulders, although tandem rollers occasionally are used. So-called "tamping" rollers for consolidating embankments of appreciable thickness generally are of the "sheepsfoot" type illustrated in Fig. 45. This roller consists of a steel drum on which projecting "feet" are mounted; the weight of the roller may be increased by filling the drum with water. Sectional rollers, consisting of heavy discs mounted on a single or separate axles, also are used.



Courtesy of Insley Mfg Co

FIG 47 Tractor-drawn shoulder finisher

Subgrading Machines and Shoulder Finishers. Subgrades and finishing shoulders on low-, intermediate-, and high-type roads are prepared by blade graders Special machines are also used for these operations and are particularly adaptable to long sections of high-type roads (See Figs. 46 and 47)

GRADING OPERATIONS

Hand Methods. Hand work in connection with grading operations on roads and streets usually is limited to the trimming of outside slopes, digging trenches, excavating for structures, and supplementing machine operations in final trimming and smoothing of slopes.

Selection of Machine Methods. The selection of grading machinery for a specific grading project will be controlled by a large number of conditions but principally by the quantity and distribution of the material to be excavated, the length and condition of haul, and requirements relative to compaction of embankments.

Formation of Grades with Blade Graders. The formation of grades with a heavy blade grader consists essentially of three operations: first, the excavation of the ditches or gutters and the deposition of the earth on the shoulders; second, the transfer of the earth from the shoulders over the entire cross-section; and third, finishing or smoothing the grade to the required cross-section.

Before starting work with the grader, a row of stakes should be set about 100 to 200 ft apart on line with the bottom of the gutter if a V-shaped section is to be used, or on line with the outside bottom edge of the gutter or ditch if a trapezoidal section is to be formed. These stakes are for the purpose of aiding the operators of the grader and tractor in making the first cut with the grader; as they are immediately destroyed, they should be relatively light so as not to interfere with the operation of the grader.

The number of ditch cuts depends upon the depth and width of the ditch or gutter to be formed, but generally two preliminary cuts, only, are necessary. These cuts are made most effectively with the blade of the grader set at an angle of approximately 60° with the longitudinal axis of the road. A later cut, with the blade swung out to cut the outside slope of the ditch, is usually made after most of the earth has been spread on the roadway and shoulders; the outside ditch slope may be cut by a backsloping attachment instead of by the blade. The first ditch cut should do little more than cut and break up the sod on the surface of the ground and transfer it to the shoulder. The second ditch cut should be made before any material is transferred toward the center of the roadway. If care then is observed by the grader operator in not setting the blade too low in moving the earth over the shoulder, the clods of sod will remain on the shoulder instead of being transferred to the roadway. The transfer of sod to the roadway is not very objectionable if a light-traffic road is being improved and the natural earth surface will be used for traffic or the construction of a low-type surface will not follow immediately after grading. The finishing of the grade will break up the clumps of sod, and the grass and other vegetation ordinarily will soon disappear.

The surface of an earth grade is usually somewhat loose and rough after shaping, and the next operation consists of smoothing it. If the earth surface is to be the wearing surface, the grade is finished to the required cross-section. When a separate wearing surface is to be placed, a flat cross-section often is formed; clay subgrades, however, should be shaped so as to drain to the sides.

Excavation with Scrapers. Generally, before excavation, the area to be excavated must be loosened, and road rooters or scarifiers attached to graders are used extensively for this purpose. Blade graders are commonly operated in connection with scrapers, and the graders themselves often may be used directly for loosening. If a large amount of side-cutting of high banks is involved, power shovels usually are desirable, but low side banks may be cut down by graders and the excavated material can then be transported to embankments by scrapers.

Borrow Excavation. On roads on which grade reduction is unnecessary, and on roads on which the volume of excavation will be insufficient, embankments are often formed by borrow excavation. Borrow pits alongside roadways are objectionable unless they are higher than the roadway and can be excavated and trimmed so as not to present an unsightly appearance afterwards. All borrow pits should be carefully cross-sectioned before and after excavation in order to determine the quantity of earth excavated.

Borrow excavation is generally necessary when objectionable subgrade soils are replaced, when blanket courses are constructed on subgrades, and on locations where the subgrade soil is unsuitable for shoulders.

Formation and Compaction of Embankments. Earth embankments should be placed in thin layers in order to secure compaction, starting with the full width of the embankment at the base and decreasing the width in accordance with the required slopes as the fill is carried up Dumping over the sides or ends of embankments is objectionable because the earth is not well compacted. Lack of bond between the old embankment and the new material may also result in slipping of the fill on sides of old narrow embankments or on hillsides. The thorough compaction of side fills, either on hillsides or on the sides of existing embankments, by means of rollers, is impracticable, and when thorough consolidation is to be obtained under such conditions, it is necessary to excavate and move the earth laterally in layers which are sufficiently thin to be compacted by means of rollers. This procedure generally is not difficult on old embankments and may often be accomplished by means of road rooters and graders or by means of graders with scarifier attachments. Compaction of embankments on sidehill locations is difficult, however, and excavation to a comparatively flat subgrade may be necessary when thorough consolidation is essential. It should be apparent that complete consolidation of embankments by artificial means is unnecessary on many locations, particularly on land-service or other roads on which only light traffic is encountered; natural settlement of embankments on such roads is not only feasible but may be necessary for reasons of economy. *When embankments are placed on sidehill locations or old embankments are widened by the addition of new material, the inclined slopes should be roughened by means of rooters or plows to provide bond between the existing surface and the new fill.*

The consolidation of embankments is secured by means of the grading machinery for forming the embankment, by natural settlement, and by rolling. Wheel scrapers, tractors, and wagons assist in compacting embankments owing to their weight and its distribution.

The artificial compaction of embankments on which medium- and high-type wearing surfaces are to be constructed is often an important feature of road construction on state and county through-highways. In some locations the selection of suitable material and the careful design of the side slopes must be considered in order to prevent excessive settlement, slides, and excessive erosion.

The compaction of the soil in an embankment depends upon the nature and condition of the soil on which the embankment will be placed, upon the soil placed in the embankment, upon the moisture content of the soil at the time it is compacted, and upon the method employed for compaction.

Unsatisfactory soils occurring below the proposed locations of embankments may be treated by several methods which will be described later in this chapter. The vertical consolidation of some soils below embankments may be hastened by drainage. In general, most good soils will show but little consolidation resulting from overlying embankments.

Sandy soils often do not require extensive compaction, but clays and mixtures of clay and sand compact well under rolling when the moisture content is properly controlled. Silty soils, unless containing sufficient cohesive clay to prevent the capillary movement of water, are generally not satisfactory for embankments.

Experimentation and experience have shown that effective compaction of good soils can be secured only when the moisture content is controlled. The optimum moisture content and the maximum density of the soil may be determined by the Proctor test described in Chapter II and are employed as a measure of the compaction secured on embankments which are artificially compacted. Sheepsfoot rollers are utilized almost universally for compaction of embankments. Tests of soils for moisture content are made, and soils containing insufficient moisture are sprinkled and soils containing an excess of moisture are allowed to dry. The thickness of the layer of soil is a very important factor and generally is restricted to a maximum of 8 in., loose measure.

The determination of safe slopes and heights of embankments is a complex problem, and engineering literature during the past several years has been filled to overflowing with theoretical studies of the stability of embankments. The results of these studies often are conflicting and difficult to apply in practice The subject is too large for treatment in this volume and the interested reader is referred to the bibliography at the end of this chapter.

Shaping of Subgrades. The economical placing of wearing courses requires the accurate shaping of subgrades. Subgrades for flexible types of wearing courses may be formed by means of blade graders for both trench and surface methods of construction. Subgrading machines often are used in connection with side forms. The shaping of subgrades will be discussed in connection with descriptions of methods of constructing various types of wearing courses and pavements.

Finishing Shoulders and Side Slopes. Shoulders and side slopes are finished by means of blade graders, special shoulder-finishing machines, and hand trimming. The use of shoulder-finishing machines generally is confined to high-type pavements. Some hand finishing usually is necessary on all grading operations, but excessive hand finishing should be avoided on account of cost

The compaction of shoulders is essential on flexible types of wearing courses to provide side support for the roadway but also is desirable on rigid pavements to support vehicles which may use the shoulder for parking or emergencies.

Excavation for Structures. Excavation for bridges, culverts, retaining walls, pipe drains, and other structures is usually a hand operation on small structures or a combination of hand and machine work on large structures Truck cranes and crawler-type cranes are generally used on bridge and culvert excavation, which may be handled from the adjacent roadway grade.

Trenching machines are adaptable to the excavation of trenches for pipe drains when the amount of excavation and the condition and contour of the ground surface are favorable to their operation. Open ditches for cross-drainage may frequently be formed by means of blade graders, but generally hand methods are used, either alone or in combination with graders and scrapers.

Seeding and Sodding. The seeding and sodding of shoulders and side slopes are practiced both to prevent erosion and to improve the appearance of highways. Both operations are expensive, and for the latter purpose they naturally have been restricted to the more important highways.

Seeding should be done as soon as possible after grading operations have been completed. Seeding should be at the rate of 60 to 70 lb per acre, and all surfaces should be lightly harrowed or raked to cover the seeds.

Sod is usually put on slopes and must be placed with a great deal of care. The sod is cut into strips about 10 to 12 in. in width by hand or by special cutters hauled by a team or tractor. Sod should be taken from soils which are similar to the soils on which it is to be placed and should be removed to the full depth of the roots. The tops and bottoms of slopes should be rounded, and the ground surface should be smoothed to the regular shape and slope. The sod should be placed in rows parallel to the center line of the road, beginning at the toe of the slopes, and each strip of sod should be lightly tamped with the flat side of a shovel to close the joints. The top edge of the last rows of sod should be turned slightly into the face of the ground and covered with a thin layer of earth, which is then compacted so as to cause surface water to run over the sod. When sod is placed on high slopes (more than 6 to 8 ft high) it is desirable to secure the sod to the earth below by means of wooden pegs about 12 in. long and 1 in. thick which are placed in every second or third layer of sod and spaced about 18 in. apart in the rows.

Mulching. Deep excavation for highways generally exposes infertile subsoil on which successful seeding, and even sodding, is difficult. The application of mulches to sterile slopes has proved effective. Many kinds of mulches have been successful, but local organic materials such as roadside mowings, weed straw, and woods litter are often the most practicable. Light applications of mulch are desirable to permit free air circulation. The mulch must remain in close contact with the soil surface, and on all but gentle slopes it is necessary to hold it in place by an overlayer of brush or poles, by means of stakes, or by placing a shovelful of tight soil on top of the mulch. During the first growing season the surfaces treated with the mulch will be covered with weeds and briars, which will be followed in two to three years by sod. Sod-forming grasses may be added to the decomposing mulches.

Preservation of Trees. Roadway excavation and embankment operations often interfere with existing roadside trees. Fills of nonporous soils over the base of trees may cut off the supply of oxygen and the escape of waste gases from the roots, and cuts may lower the water table to such an extent that an insufficient supply of moisture is left for dissolving and transporting the minerals which are essential for the continued life of the trees. In many cases it is possible to preserve trees by artificially controlling the air and water in the soil.

Trees at or near the tops of excavation slopes may be protected by constructing a rock wall to prevent erosion, to retain sufficient moisture for tree growth, and to permit aeration and drainage of the root systems. Any clay fill over the base of trees is detrimental. Some trees, notably poplar, elm, and locust, will withstand sand fills up to 12 in. in depth, but others, including maple, oak, birch, beech, and hickory, appear to be sensitive to even very shallow fills. The favored method for fills consists of building tree wells or placing a coarse rock fill around the tree.

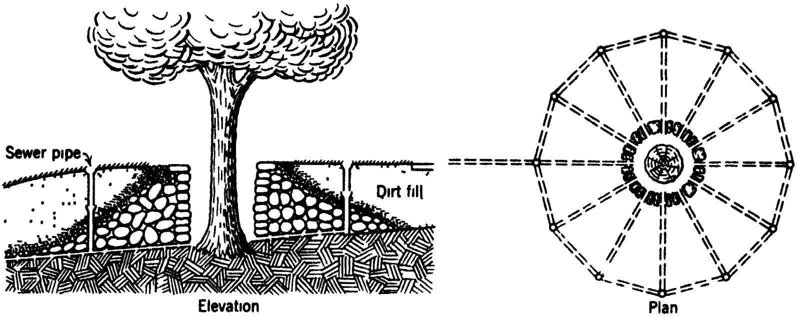


FIG 48 Details of a tree well for preservation of a tree on a proposed fill. (*A Robert Thompson*)

A tree well is constructed by first placing drain tile or perforated metal pipe on the original ground surface around the tree, with vertical bell-end tile or sewer pipe at intervals extending to the new ground surface, as shown in Fig. 48. A cone of coarse stone is then placed over the tile or pipe and a dry well built up with coarse stone about 20 to 30 in from the trunk of the tree. A layer of fine stone or gravel is placed on the rock cone, covered with a layer of hay or straw, and the fill then is completed to grade. The dry well may be left open or filled with coarse sand or fine gravel and charcoal. Sometimes the tree well is omitted and the rock cone placed up to the trunk of the tree with its top at the new ground surface. If the natural ground surface has a good slope, the tile is sometimes omitted.

Classification and Measurement of Excavation. Grading operations on highway improvement generally involve excavation for various purposes, and, on contract work, at unit prices, the grading quantities are usually separated into several items according to the nature of the work. The following items are representative of this practice:

Roadway excavation.
Channel excavation.
Excavation for structures.
Ditch excavation (usually applied to ditches for cross-drainage).
Borrow excavation.
Machine grading (applied to the formation of roadways and shoulders by means of blade graders).
Overhaul of excavation.

When two or more different materials are encountered on projects to be improved under the contract system and in such quantities as to render it necessary to perform, on a somewhat large scale, more than one distinct type of operation, it is desirable to classify the materials into separate contract items. Any method of classification must necessarily be arbitrary, there being no scientific distinction between the various materials of excavation. In general, the classifications which have been adopted have been based on the difficulty of loosening and on the volume of consolidated masses, and include two or three separate classes: earth (or common) excavation, loose rock, and solid rock; or merely earth excavation and rock excavation. The line between earth and loose rock excavation is very difficult to define. The following specification illustrates an unsatisfactory definition of earth excavation:

Earth excavation shall include all boulders, ½ cu yd or less in volume; all deposits of gravel, sand, loam; clay layers, veins, or seams; veins or layers of ferruginous cemented sand; peat and marl deposits; soft and decomposed deposits of sandstone, shale, or schist; and all other materials that can be removed without blasting by hand or with a power shovel of ½-cu-yd capacity that is in good working condition.

It now is common practice to define rock excavation or solid rock excavation and loose rock excavation quite carefully and specify that "all material not classed as loose or solid rock (or as rock) shall be classed as earth (or common) excavation." The following definitions of solid rock excavation, loose rock excavation, and rock excavation are representative of present practice:

Solid rock excavation shall comprise rock in solid beds or masses in its original position which requires blasting for removal, and boulders or detached masses of rock having a volume of 1 cu yd or more.

Loose rock excavation shall comprise all detached masses of rock or boulders having a volume of more than 1 cu ft and less than 1 cu yd, and all other rock which can be properly removed by barring and picking, although power shovels or blasting may be used to facilitate the work.

Rock excavation shall comprise rock in solid beds or masses in its original position which requires blasting for removal, and boulders or detached masses of rock having a volume of 13 cu ft or more.

SWAMP FILL CONSTRUCTION

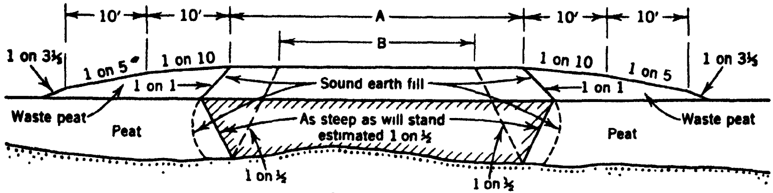
Swamps, peat bogs, and "sinkholes" often present a perplexing problem to highway engineers and administrative officials. The nature and composition of the materials encountered in such areas were described in Chapter II. Unless the condition extends over only a very small space, the most satisfactory solution consists of the relocation of roads around it; relocation, however, may be impracticable, and various methods are followed for constructing and consolidating fills on these locations. These methods consist of gravity subsidence (or natural settlement) of fills, excavation of the unsatisfactory material, displacement of the unstable material, or various combinations of these processes.

Gravity Subsidence. The consolidation of embankments over soft subsoils by natural settlement and subsidence has been practiced since early times. In many instances, final settlement may require long periods of time, and for this reason gravity subsidence is not satisfactory for roads on which rigid pavements or costly types of flexible pavements are required. Gravity subsidence is not entirely objectionable on light-traffic roads, particularly if arrangements are made to add material to the fills at more or less regular intervals to take care of the subsidence. The total cost over a long period of years may be large but may not exceed the cost of other methods, and gradual filling possesses the advantage of utilizing normal annual budgets rather than concentrating a large expenditure in a single year.

Excavation Methods. Complete excavation of peat and muck subsoils is practiced on roads on new locations on which costly types of wearing surfaces are required. The excavation of stable soil material in existing embankments, unless very small in amount, is usually impracticable, and other methods, which will be described later, are adopted. Complete excavation is considered practicable up to depths of 8 to 15 ft and may be practicable for greater depths on very short lengths of fills.

Cranes equipped with clam shell or dragline buckets are generally used for excavating, and filling should accompany excavation to prevent the remaining peat or muck from moving in from the sides to refill the excavated area. A typical cross-section of a fill constructed by this method is shown in Fig. 49.

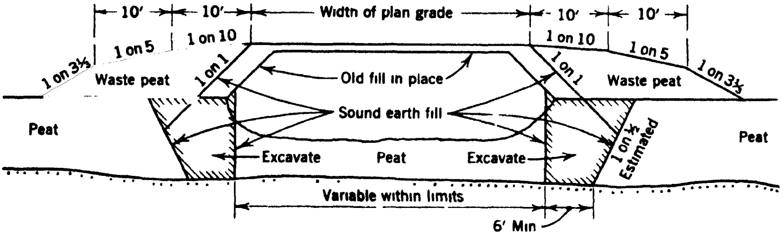
Existing fills of large size and possessing much stability are often widened by excavating and backfilling trenches at the side, as illustrated in Fig. 50. The backfilling proceeds simultaneously with the excavation as in the previously described case. This procedure is applicable to comparatively shallow depths, and the side trenches may be excavated completely or only partly.



Courtesy of Michigan State Highway Department.

FIG. 49. Typical cross-section for total excavation through shallow peat swamps.

Displacement Methods. Excavation of deep layers of muck and peat by machinery is impracticable, and the construction and consolidation of fills on such areas may be accomplished by displacing the soft subsoil by jetting with water or by means of explosives. This procedure is particularly adaptable to locations on which an existing fill must be widened and consolidated; it is illustrated in Fig. 51.

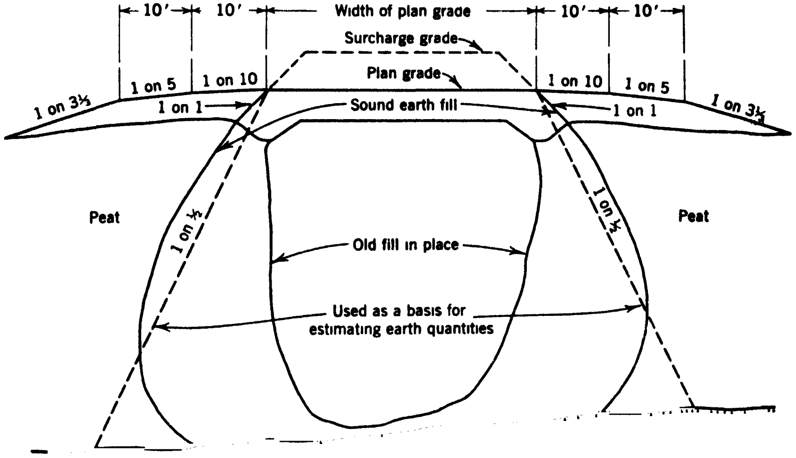


Courtesy of Michigan State Highway Department.

FIG. 50. Typical cross-section for partial excavation through shallow peat swamps when stability and size of existing fill make total excavation impracticable.

Jetting is performed by pumping water through a series of pipes which are placed in bored holes along the embankment to be settled. Jetting pipes are usually placed in two rows, one on each side of the embankment, are spaced from 10 to 25 ft apart in the rows, and extend into the peat or muck for about two-thirds of its depth. The filling proceeds simultaneously with the jetting, and the jetting usually covers an area extending about 40 to 75 ft in front of the new fill. Side trenches may be excavated along the sides of the old fill to accelerate the displacement of the soft material. Jetting also is practiced on deep fills on new locations. This method, of course, is possible only when a large supply of water is available. Ten to fifteen jets are generally operated at the same time.

Blasting frequently is substituted for jetting but is somewhat less effective because it cannot be as completely controlled. The blasting procedure may involve preliminary blasting of the peat or muck before filling, which is followed by deep blasting after placing the fill. Preliminary blasting sometimes is omitted, and on locations having old fills in place blasting may be confined to the sides of the old embankment.



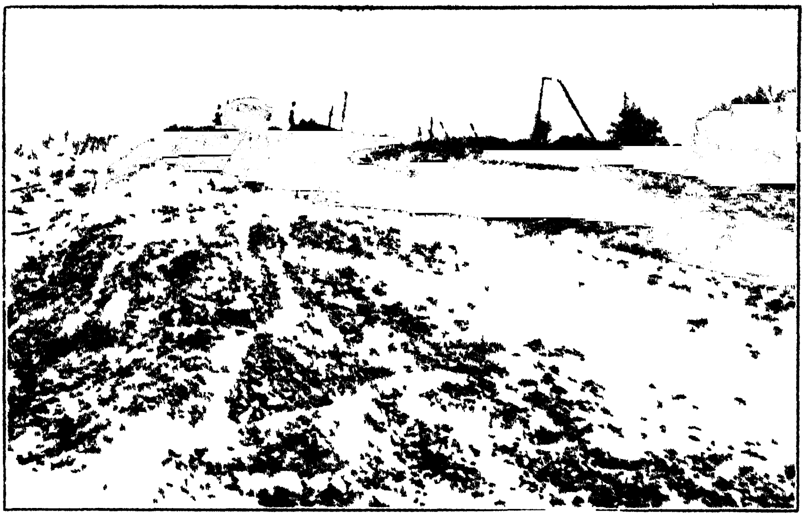
Courtesy of Michigan State Highway Department

FIG 51. Condition of highway embankment through a deep swamp where jetting or blasting may be used to stabilize the fill

Displacement methods have been successful for constructing fills up to 50 ft or more in depth.

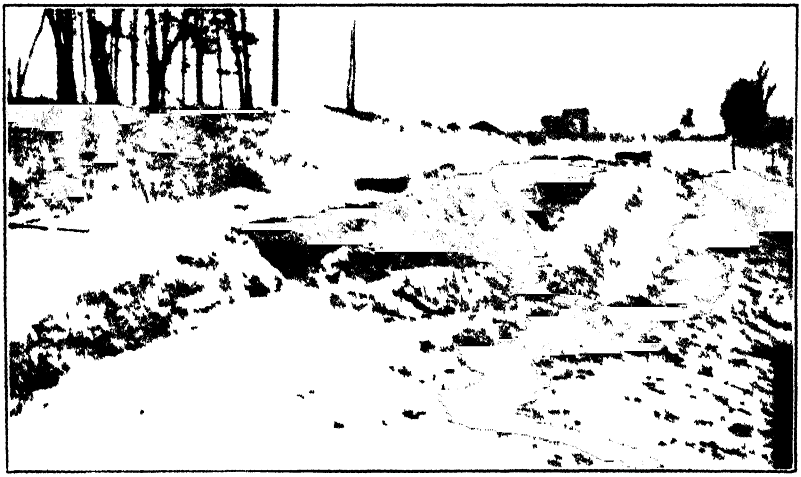
Nature of Filling Material. The selection of suitable material for the construction of embankments over swamps and peat bogs is a very important feature of the work. Clay softens in contact with water and tends to flow laterally; jetting operations accelerate this lateral flow. Sandy and gravelly soils, therefore, are desirable.

Combination Methods. Numerous combination methods which utilize gravity subsidence and partial excavation are also used. Excavation of the surface mat, only, followed by filling, is effective for shallow depths but should be practiced only on light-traffic roads.



Courtesy of Michigan State Highway Department

FIG. 52 Jetting a highway embankment over soft subsoil

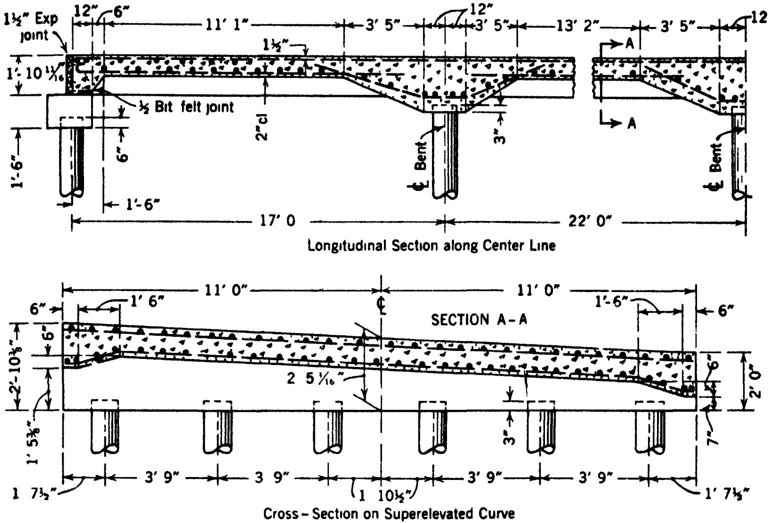


Courtesy of Michigan State Highway Department

FIG. 53 Displacement of soft subsoil below highway embankment by blasting

Partial excavation and gravity subsidence are adaptable in locations near buildings and bridges where jetting or blasting might result in damage to these structures.

Pile Bent Foundations for Rigid Pavements. The Division of Highways of the Illinois Department of Public Works has employed pile bent foundations for supporting concrete pavements across peat bogs and swampy areas, and a typical design is shown in Fig. 54. Generally, expansion joints are provided every 100 ft, and the pavement is continuous between the joints, being supported on pile bents 17 ft apart at the ends and at intermediate distances of 22 ft. The end of each adjacent slab of the pavement placed on the subgrade is supported on a cap on the end bent cast separately from the pavement, and the thickness of each adjacent slab is increased, in a distance of 7 ft, from the regular slab thickness to the thickness of the pile-supported slab at the cap center.



Courtesy Illinois Division of Highways

FIG 54 Design of pile bent foundation for concrete pavement across swampy areas

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CHAPTER VI

BITUMINOUS MATERIALS

Bitumen, an organic material of very complex structure, is defined by the American Society for Testing Materials as "a mixture of native or pyrogenous hydrocarbons and their non-metallic derivatives, which may be gases, liquids, viscous liquids, or solids, and which are soluble in carbon disulphide." The term bituminous material is used to describe materials consisting principally of bitumen or containing bitumen. Bituminous materials are valuable for various highway and street purposes because of their cementing and waterproofing properties. The bituminous materials used in the improvement and maintenance of roads and streets consist of asphalt products, which are obtained from natural asphalt deposits or are produced from petroleum, and tar products, which are obtained in the manufacture of gas or coke from bituminous coal and in the manufacture of carburetted water-gas from petroleum distillates.

SOURCES OF ASPHALT

Native Asphalt. Native asphalt is found in lakes, in land deposits resulting from the overflow of lakes, in springs which exude soft bitumens, in fissures in the earth which contain a hard and brittle asphalt, and in rock deposits in which the asphalt impregnates the rock. Rock asphalt is not used as a source of asphalt, and its occurrence, production, and uses will be described separately. Several theories have been advanced to explain the origin of native asphalt but will not be discussed here except to state that there is a strong probability that native asphalt was formed from asphaltic petroleum under the action of heat, pressure, and evaporation.

The largest and most important sources of native asphalt are the island of Trinidad and Venezuela. Both land and lake deposits are found on the island of Trinidad, but the lake deposit is used almost entirely. The crude Trinidad lake asphalt is a uniform mixture of water, gas, fine sand, clay, and bitumen, the bitumen content being about 39 per cent. Refined Trinidad lake asphalt contains approxi-

mately 57 per cent bitumen and 43 per cent mineral matter and is too hard for direct use; it is softened by mixing with a softer petroleum asphalt residue. Bermudez asphalt occurs in a shallow lake in Venezuela and is much more variable in composition than Trinidad lake asphalt, the bitumen content ranging from about 45 to 88 per cent. Most of the impurities are removed in refining, and the refined asphalt usually contains from about 92 to 97 per cent of bitumen. Refined Bermudez asphalt is softer than Trinidad asphalt but is too hard for most road and street uses and is softened by mixing with a softer petroleum asphalt.

Other native asphalts include gilsonite and grahamite, which are found in rock fissures, principally in Utah and Oklahoma, respectively, and Cuban asphalt. Gilsonite and grahamite are used less extensively now than in former years; gilsonite is used to some extent in cold-laid paving mixtures. Native asphalt from various local sources is used in Cuba for hot-mix asphalt pavements and is fluxed to the desired consistency.

Petroleum Asphalt. Petroleums vary widely in their chemical composition and physical characteristics and are usually classified as asphaltic petroleums, which yield asphaltic residues, and as paraffin petroleums, which yield semi-solid paraffins with coke as the final residual product. Asphaltic petroleums from Mexico and California once were used exclusively to produce asphaltic products for road and street purposes, but many other petroleums now serve as sources of these materials.

SOURCES OF TAR

Manufacture of Coal Tar. In the manufacture of coal tar, the bituminous coal is charged into retorts or ovens where it is heated to high temperatures which drive off the volatile matter in the coal. The gases thus driven off are conducted by means of pipes to a closed trough, known as a hydraulic main, which is partly filled with water. A large percentage of the tar condenses out in the water in this trough and is removed. The gases pass on to a condenser, which may be cooled by either air or water. More tar condenses here and is drawn off. The gases are then passed through a device known as a scrubber, of which there are several varieties. By means of water sprays operated in connection with screens or baffle plates, the rest of the tar is extracted and is drawn off from the bottom of the scrubber.

Manufacture of Water-Gas Tar. Water-gas is manufactured by allowing steam to come into contact with anthracite coal or coke which has been heated to the point of incandescence. Water-gas is *non-luminous, and when illuminating gas is desired it is enriched with a petroleum distillate,* the product known as gas-oil being usually employed. The oil is sprayed into a carbureter where it mixes with the water-gas and is vaporized. The mixture of gas and oil then passes into a chamber partly filled with brick, which is known as a super-heater and in which the temperature is maintained high enough to "crack" the oil. The lighter hydrocarbons formed in this process are gaseous and burn with a luminous flame. Heavy products are also formed and are later condensed to form water-gas tar.

REFINING PROCESSES

The principal refining processes in connection with the manufacture of bituminous materials are sedimentation, dehydration, distillation, fluxing, emulsification, and air-blowing.

Sedimentation. Native asphalts and crude petroleums contain some water and mineral and vegetable matter. These impurities may be removed to a large extent by merely allowing the material to settle so that a mechanical separation, due to differences in specific gravity, occurs. The heavier native asphalts must be heated to render them sufficiently fluid to enable sedimentation to take place.

Dehydration. Some materials do not readily eliminate water by sedimentation and it is necessary to remove the water by the application of heat. The various types of dehydrating plants will not be described in detail. Two principal devices are used. In one, known as a topping plant, the crude material is pumped through heated pipes which change the water into steam as the material is discharged, allowing the heavier bitumen to be collected separately. In another type of plant, for dehydrating tars, the crude material passes over heated baffle plates which separate the water and light oils from the heavier tar.

Fractional Distillation. Fractional distillation consists of the mechanical separation of the various products or "fractions" in the original materials without chemical change. Both crude asphalt-bearing petroleums and tars are refined by this process. Two products are formed in the distillation process: distillates, which are vaporized and are collected by condensing; and residues, which are heavier products and remain in the still. Both batch and continuous processes of distillation are employed in the production of asphalt products.

Destructive Distillation. Destructive distillation involves the removal of all the lighter constituents in the crude material until only coke is left behind in the still. This process is carried out when large quantities of distillates are desired.

Cracking. Cracking may be described as that process of distillation which causes the constituents of the material in the still to break up into entirely different substances; in other words, chemical change takes place. Cracking is most effectively produced by distilling under pressure, although it will occur to a lesser extent if the material in a still not under pressure is not agitated mechanically or by steam. The cracking process increases the yield of the lighter distillates, which are usually the most valuable fractions of the original material. Residues from the cracking process formerly were infrequently used for highway purposes, but cracked liquid residual products are now used in very large quantities, and heavier residues to some extent in the manufacture of cut-back asphalts.

Emulsification. Asphalts are not soluble in water, but emulsions of asphalt and water serve many road and street purposes. The water evaporates and leaves the asphalt in place, and the action of emulsions, therefore, is very similar to that of cut-back asphalts. Asphalt emulsions are suspensions of very small globules of asphalt in water which depend for their stability on emulsifying agents. The commonest emulsifying agents are soaps and an alkaline liquor.

Some emulsions may be produced in a tank by agitation during mixing, but others are very unstable when prepared in this way and are produced in high-speed disintegrators or colloid mills; the heated asphalt and the solution or mixture of water and emulsifying agent are fed simultaneously between closely spaced revolving discs or conical surfaces which break up and disperse the asphalt through the water.

Asphalt emulsions separate or "break down" when placed in contact with mineral aggregates or with road and street surfaces. The rate at which this separation takes place may be controlled in the manufacturing process, and the resulting products are described as *quick-breaking, medium-breaking, slow-breaking,* or *mixing* emulsions.

Fluxing. Fluxing consists of softening a heavy product by the addition of a lighter or softer one and is a process in making many asphalt products. Both residual products and distillates serve as fluxes; the resulting material is called fluxed asphalt when a residual product is the flux, and "cut-back" asphalt when a distillate is the flux.

Residual petroleum asphalts are refined to the consistency required for the various grades of asphalt paving cements and therefore do not require fluxing. Refined native asphalts, however, are too hard to use directly as paving cement and are fluxed by adding residual petroleum asphalt. The fluxing operation in connection with native asphalts for paving cements was at one time commonly performed at the paving plant in kettles in which the two materials, after melting, were mixed by agitation with steam or mechanical agitators. The fluxing of native asphalts now is generally conducted at the refining plant.

The production of "cut-back" asphalts by fluxing a residual asphalt with a distillate is performed at asphalt refineries by both batch and continuous processes. The function of the distillate in cut-back asphalts is temporary and largely to facilitate their application to road surfaces or mixing of the asphalt with mineral material; the distillate evaporates, leaving the residual asphalt in place. *Rapid-curing* cut-back asphalts are produced with gasoline or naphtha for the flux; *medium-curing* cut-back asphalts are produced with kerosene for the flux.

Air-Blowing. The blowing process consists of forcing air through heated asphalt. The air enters the asphalt through perforated pipes in the bottom of stills and must be of sufficient quantity to agitate the asphalt violently. The oxygen of the air unites with some of the hydrogen from some of the hydrocarbons to form water which is carried off as vapor. Some oxygen may also combine, chemically, with some of the hydrocarbons. Inasmuch as there is a removal of hydrogen, and perhaps an addition of oxygen, the process generally has been described as oxidation, but some chemists doubt the addition of oxygen and object to the term "oxidation" as descriptive of the process. The blowing of asphalt permits the recovery of much larger proportions of residual asphalt than may be obtained by fractional distillation because of the chemical changes which occur. The most important changes in physical characteristics are increased hardness (which may be measured by the tests for penetration and softening point) and a decrease in ductility and cementitiousness.

ROCK ASPHALT

Occurrence. Rock asphalt usually consists of limestone or sandstone which has been naturally impregnated with asphalt or maltha and is found in many areas in Europe, Canada, and the United States. Rock asphalt from the island of Sicily was used for paving in coast cities of the United States prior to the development of modern types of bituminous pavements. The best known deposits of rock asphalt in the United States are the sandstone rock asphalt deposits of Kentucky and the shell-limestone rock asphalt found in Uvalde County, Texas. Rock asphalt also is found in Oklahoma, Alabama, and other states.

Mining and Preparation of Rock Asphalts. Rock asphalt is excavated like rock by quarrying or mining. The excavated material is crushed into sizes suitable for use as paving mixtures. Both types of rock asphalt vary in bitumen content in different parts of the deposits, and the material from various parts of each deposit usually is carefully blended to secure the desired bitumen content and the required uniformity in bitumen content. Some rock asphalts are low in bitumen content and may be enriched by the addition of suitable grades of asphalt products; others contain bitumen which is too hard, and light residual oils or fluxes are added to soften it. In some cases, mineral aggregates are added to increase the resistance of the material to skidding after placing in a pavement or on a road or street surface.

TABLE 3

TYPICAL ANALYSES OF ROCK ASPHALTS

	1	2	3	4
Bitumen, per cent. . .	6.4	7.1	10.3	6.9
Passing No. 200 sieve, per cent	6	14	8	3
Passing No. 100 sieve, per cent	4	22	3	28
Passing No. 80 sieve, per cent	4	5	3	24
Passing No. 50 sieve, per cent	30	8	6	16
Passing No. 40 sieve, per cent	22	2	5	2
Passing No. 30 sieve, per cent	5	2	7	1
Passing No. 20 sieve, per cent	2	2	9	1
Passing No. 10 sieve, per cent	3	6	17	3
Retained No. 10 sieve, per cent	18	32	32	15

CHARACTERISTICS OF BITUMEN

	1	2	3	4
Specific gravity 25° C/25° C	1.022	1.033	1.088	
Softening point, degrees F	96	123	185	
Penetration at 25° C, 100 grams, 5 sec	242	60	4	
Ductility at 25° C, cm	Too soft	91	0.5	100+
Float test at 50° C				113

(1) Breckenridge County, Kentucky (2) Cherokee, Alabama. (3) Uvalde County, Texas. (4) Murray County, Oklahoma

Characteristics of Rock Asphalt. Table 3 contains typical analyses of several rock asphalts.

PROPERTIES AND TESTS OF BITUMINOUS MATERIALS

The tests for determining the characteristics of bituminous materials may be classified as consistency tests, heat tests, solubility tests and tests to determine composition, and weight tests.

Most of the common tests of bituminous materials have been standardized by the American Society for Testing Materials or the American Association of State Highway Officials, and the details of testing procedure will be found in the published standards of these organizations; descriptions of tests and properties in this discussion will be confined to the general principles involved and to the applications of the various tests.

Consistency Tests

Consistency. The consistency of any bituminous material refers to its degree of fluidity or solidity and determines its adaptability to various methods of handling and utilizing.

Penetration. The penetration of a bituminous material refers to the depth of penetration of a standard needle when applied to the surface of the material under standardized conditions of test. The penetration test for consistency is conducted on semi-solid asphalt products, such as asphalt cement, for use in bituminous macadam, asphaltic concrete and sheet asphalt, fillers for block pavements, and residues resulting from heat tests. The penetration test is not well adapted to semi-solid tar products because of their high surface tension and adhesiveness. The test ordinarily is conducted under the following standard conditions:

200 grams,	60 seconds,	at 32° F
100 grams,	5 seconds,	at 77° F
50 grams,	5 seconds,	at 115° F

The standard test at 77° F customarily serves as an arbitrary measure of consistency. The tests at 32° F and 115° F are of value in measuring the "susceptibility" of asphalts to temperature changes.

Float Test. The float test for bituminous materials refers to the time required to melt or soften a plug of the material in water at a given temperature under standardized conditions of test. This test is useful for determining the consistency of tars.

Viscosity. The viscosity of a bituminous material refers to its rate of flow through a given orifice under standardized test conditions. The test is useful in measuring the consistency of materials that are too soft for determination of penetration and is applicable to both asphalt and tar products. The viscosity determination is of greatest value when conducted at or near the temperature at which the material will be used. Among the several types of viscosimeters which are utilized are the Engler, the Saybolt Universal, and the Saybolt Furol. The quantity of material employed is commonly 50 ml with the Engler viscosimeter and 60 ml with the Saybolt viscosimeters, and the viscosity is measured by the number of seconds of time consumed in the flow of the specified quantity of material. The "specific viscosity" of some materials sometimes is described in specifications and refers to the ratio of the time of flow of the bituminous material to the time of flow of the same quantity of water at a temperature of 77° F Standard temperatures commonly employed are 77° F, 100° F, 122° F, 140° F, 180° F, and 210° F, particularly with the Saybolt visosimeters. A temperature of 104° F (40° C) sometimes is employed with the Engler viscosimeter.

Ductility. The ductility of a bituminous material refers to the distance which it may be drawn out or stretched under standardized conditions of test. The test is usually applied to asphalt cements for use in bituminous macadam, in asphaltic concrete and sheet asphalt, and in fillers for block pavements. Most, but not all, petroleum asphalts prepared by straight distillation have high ductility (over 100 cm) at normal temperature (77° F). The ductility of oxidized asphalts, utilized for fillers, is generally rather low at normal temperature but is not so much affected at lower temperatures. The test is employed as a control test by some refineries and, by users, principally to determine the degree of air-blowing.

Softening Point. The softening point, or melting point, as it is sometimes called, refers to the temperature at which a bituminous material softens under given conditions of test. Bituminous materials differ from many other substances in that there is no critical temperature at which the change from a solid to a liquid state takes place, and the determination is therefore entirely arbitrary. For this reason the term "softening point" is preferable to "melting point."

The determination of softening point is valuable in connection with asphalt and tar products intended for use as seal coats, fillers, and binders for bituminous macadam. When employed along with the penetration test, the softening-point determination is a measure of the "susceptibility" of bituminous materials to temperature changes; when two asphalts have the same penetration at 77° F, the one having the higher softening point generally is the less susceptible.

HEAT TESTS

Flash Point. The flash point, as its name implies, refers to the temperature at which a material will flash when brought into contact with a flame. The flash-point test identifies various types of bituminous materials and determines the temperature conditions under which they may safely be used. The test is applicable to both asphalt and tar products.

Volatilization Test. The volatilization test is conducted by subjecting the bituminous material to heat under standardized test conditions. The loss in weight of different materials varies. Highly fluid materials show high losses, and semi-solid residual products low ones. The test is therefore a means of identification. It is utilized for that purpose, to determine the suitability of various fluid products for dust palliatives and surface applications, and to determine the degree of hardening of semi-solid asphalts under the action of heat. The test is regularly conducted on asphalt products. On tars, the distillation test takes its place.

The value of the standard volatilization test, which is conducted on a 50-gram sample in a flat-bottom, cylindrical container 2.17 in. in diameter and 1.38 in. in depth, for determining the effect of heating on the hardening of asphalts under actual construction conditions is doubtful, and R. H. Lewis and J. Y. Welborn of the U. S. Public Roads Administration have shown that a better indication of the effect of processing operatings in paving will be obtained from an oven test on samples which are only ⅛ in. in thickness.

Distillation Test. The distillation test is similar in principle to the volatilization test but is conducted under different conditions. The loss in weight at different temperatures is determined. This is one principal point of difference from the volatilization test, in which only one loss is determined. The test is widely used for testing tar products and cut-back asphalts, and serves as a means of determining their suitability for various purposes. The test also determines the extent of the distillation to which crude tar has been subjected. It also is applicable to asphalt emulsions. The procedure of making the distillation test varies to some extent for tar and asphalt products.

TESTS FOR SOLUBILITY AND COMPOSITION

Solubility in Carbon Disulphide. The extent of solubility of bituminous material in carbon disulphide determines the percentage of bitumen which is present. Petroleum asphalt is almost pure bitumen. Natural asphalts contain considerable percentages of organic and inorganic matter insoluble in carbon disulphide. Tar contains very little mineral matter, but the insoluble organic matter may vary from less than 1 per cent to 40 per cent. The insoluble organic matter in tar consists of free carbon which is present in the form of very small particles uniformly distributed.

Solubility in Carbon Tetrachloride. Petroleum and natural asphalts are normally nearly as soluble in carbon tetrachloride as in carbon disulphide. Asphalt products that have been damaged by overheating usually show lower solubility in carbon tetrachloride, and the determination is therefore a measure of the care exercised in refining. The test is not used for tars.

Asphalt Content of Liquid Asphalt Products. This test occasionally is employed as a measure of the consistency and composition of residual fluid asphalt products. It refers to the proportion of asphalt cement of 100 penetration (at 77° F) which is present, and it is conducted by heating the asphalt product until the residue after evaporation has a penetration of exactly 100 at 77° F. It is used in connection with slow-curing liquid asphalts.

WEIGHT AND DENSITY TESTS

Specific Gravity. The specific gravity of bituminous materials is determined by comparing equal weights of the material and water. This property is useful in identifying the source of liquid, semi-solid, and solid bituminous materials, and in making calculations of the density of compacted paving mixtures.

MISCELLANEOUS TESTS

Oliensis Spot Test. This test is applicable only to asphalt products. It is conducted by dissolving a small quantity of the asphalt in a standard naphtha, under standardized test conditions, and applying a drop of the solution to the surface of filter paper or glass. The formation of a brown or yellowish-brown stain with a darker solid or annular nucleus in the center is reported as a positive spot. Asphalts giving positive spots are described as heterogeneous. A uniformly brown stain is reported as a negative spot, and asphalts giving negative spots are described as homogeneous This test was developed to serve as a measure of homogeneity, particularly in products manufactured by the cracking process which generally produce a positive spot Asphalts from some crude sources, produced by non-cracking processes, may also show a positive spot test, and the value of the test may be questionable in such cases Winterkorn and Eckert have presented data which show that the spot test is a means of determining the colloidal stability of bitumens and that a more complete indication of stability will be obtained with more than one solvent and possibly more than one solvent ratio

Tests of Emulsified Asphalt. A number of special tests employed for controlling the characteristics of emulsified asphalt include tests for settlement, demulsibility, residue on a No 20 sieve, effect of freezing miscibility with water, and ability to coat stone particles

Tests of Bituminous Paving Mixtures. Numerous tests are made of paving mixtures consisting of bituminous material and mineral aggregates, they include analyses of the mixtures, stability and density of the compacted mixtures, and tests of the bituminous material after extraction from the mixtures These tests will be described in Chapter XIII

ASPHALT PRODUCTS

Classification. Asphalt products consist of liquid materials which are so soft or fluid at normal temperature (77° F) that their consistency cannot be determined by the penetration test, semi-solid materials or asphalt cement, which has a penetration at normal temperature of more than 10 under a load of 100 grams applied for 5 seconds and less than 350 under a load of 50 grams applied for 1 second, and solid or hard asphalt, which has a normal penetration (100-gram load, applied for 5 seconds, at a temperature of 77° F) of 10 or less

Liquid Asphalt Products. Liquid asphalt products include residual products, mixtures of residual products and distillates, and emulsions Residual oils produced by straight (fractional) distillation are non-volatile or "slow-curing," that is, the lighter constituents evaporate slowly Some liquid asphaltic products which are similar to straight distilled products are produced by mixing residual products with heavy distillates

"Rapid-curing" cut-back asphalts are produced by fluxing semi-solid asphalt (asphalt cement) with gasoline or naphtha "Medium-

curing" cut-back asphalts are produced by fluxing semi-solid asphalt *with kerosene.*

Slow-curing liquid asphalts are used as dust layers, as admixtures for soils, and as binders for some road-mix types of wearing surfaces. Medium-curing cut-back asphalts are used for prime coats on road surfaces prior to surface treatment or construction of wearing surfaces, as admixtures for soils, and as binders for road-mix and low-cost plant-mix types of wearing surfaces. Rapid-curing cut-back asphalts are used for seal coats on road surfaces, for road-mix and plant-mix types of wearing surfaces, and, to some extent, for bituminous macadam pavements. Specification limits for slow-curing, medium-curing, and rapid-curing products are given in Tables 4 and 5.

Emulsified Asphalt. Quick-breaking or quick-setting emulsified asphalts are employed for bituminous macadam pavements, bituminous macadam patches, and bituminous surface treatments consisting of carpet coats. Medium-breaking or medium-setting emulsified asphalts are utilized principally for road-mix and low-cost plant-mix types of wearing surfaces, and for coarse-graded asphaltic concrete mixtures for repair work. The uses of slow-breaking or slow-setting emulsified asphalts include fine-graded asphaltic concrete mixtures, sheet asphalt mixtures, and as admixtures for stabilizing subgrade soils. Table 6 gives specification limits for various emulsified asphalts.

Asphalt Cements. Asphalt cements are produced by straight (i.e., fractional) distillation with or without air-blowing. The principal applications of asphalt cement are for mixtures with mineral aggregates in hot-laid types of wearing courses or pavements such as asphaltic concrete and sheet asphalt, but it also is utilized in the construction of seal coats, bituminous surface treatments (of the carpet coat type), and for filling cracks and joints. On account of the wide range in the consistency of asphalt cements, the term "soft asphalt cement" is widely used to describe those grades having a normal penetration of 100 or more. The Division of Simplified Practice of the National Bureau of Standards has established standard grades of asphalt cement based on normal penetration, as follows:

25–30
30–40
40–50
50–60
60–70
70–85
85–100
100–120
120–150
150–200

Tables 7 and 8 contain specifications for asphalt cements.

TABLE 4

Specifications for Slow-Curing Liquid Asphalt Products

(The Asphalt Institute)

Grade Designation	SC-0	SC-1	SC-2	SC-3	SC-4	SC-5
Characteristics						
Water, per cent	0 5–	0 5–	None	None	None	None
Flash point, Cleveland open cup ° F	150+	150+	175+	200+	225+	250+
Furol viscosity						
At 77° F	75–150					
At 122° F		75–150				
At 140° F			100–200	250–500		
At 180° F					125–250	300–600
Total distillate to 680° F, per cent, by volume	15–40	10–30	5–25	2–15	10–	5–
Residue from distillation						
Float test at 122° F	15–100	20–100	25–100	50–125	60–150	75–200
Asphalt residue of 100 penetration, per cent	40+	50+	60+	70+	75+	80+
Ductility of asphalt residue at 77° F	100+	100+	100+	100+	100+	100+
Solubility in carbon tetrachloride, per cent	99 5+	99 5+	99 5+	99 5+	99 5+	99 5+
Application temperature, ° F	50–120	80–125	150–200	175–250	175–250	200–275

Note If the material fails to meet the requirement for solubility it will be acceptable if its solubility in carbon disulphide is 99+ per cent, and the proportion of bitumen soluble in carbon tetrachloride is 99 65+ per cent

TABLE 5

Specifications for Medium-Curing and Rapid-Curing Liquid Asphalt Products

(*The Asphalt Institute*)

Type of Product	Medium-Curing Products						Rapid-Curing Products					
Grade Designation	MC–0	MC–1	MC–2	MC–3	MC–4	MC–5	RC–0	RC–1	RC–2	RC–3	RC–4	RC–5
Characteristics												
Water, per cent	None	None	None	None	None	None	None	None	None	None	None	None
Flash point, Cleveland open cup, ° F	100+	100+	150+	150+	150+	150+	....	....	80+	80+	80+	80+
Furol viscosity												
At 77° F	75–150	....	.....	.....	.....	.....	75–150	....	.....	.....	.....	.....
At 122° F	.....	75–150	....	.....	.....	.....	....	75–150	.....	.....	.....	.....
At 140° F	.....	.....	100–200	250–500	.....	.....	....	....	100–200	250–500	.....	.....
At 180° F	.....	.....	.....	.....	125–250	300–600	....	....	.....	.....	125–250	300–600
Per cent of total distillate to 680° F by volume												
To 374° F	.....	.....	.....	.....	.....	.....	15+	10+	.....	.....	.....	.....
To 437° F	25–	20–	10–	5–	0	0	55+	50+	40+	25+	8+	.....
To 500° F	40–70	25–65	15–55	5–40	30–	20–	75+	70+	65+	55+	40+	25+
To 600° F	75–93	70–90	60–87	55–85	40–80	20–75	90+	88+	87+	83+	80+	70+
Residue from distillation to 680° F												
Per cent, by volume, by difference	50+	60+	67+	73+	78+	82+	50+	60+	67+	73+	78+	82+
Penetration at 77° F, 100 g., 5 sec	120–300	120–300	120–300	120–300	120–300	120–300	80–120	80–120	80–120	80–120	80–120	80–120
Ductility at 77° F	100+	100+	100+	100+	100+	100+	100+	100+	100+	100+	100+	100+
Solubility in carbon tetrachloride, per cent	99.5+	99.5+	99.5+	99.5+	99.5+	99.5+	99.5+	99.5+	99.5+	99.5+	99.5+	99.5+
Application temperature, ° F	50–120	80–125	150–200	175–250	175–250	200–275	50–120	80–125	100–175	150–200	175–250	175–250

Note. If the penetration of the residue from distillation is more than 200 and its ductility at 77° F is less than 100, the material will be acceptable if its ductility at 60° F. is 100+.

TABLE 6

SPECIFICATIONS FOR EMULSIFIED ASPHALT

(*The Asphalt Institute*)

Intended Use of Material	Bituminous Macadam; Bituminous Surface Treatment	Open-Graded Coarse Aggregate Road Mixes	Open-Graded Coarse Aggregate Plant Mixtures	Bituminous Concrete Mixtures		Fine Aggregate Mixes; Soil Admixtures
				Coarse-Graded	Fine-Graded	
Character of Material	Rapid Setting	Medium Setting			Slow Setting	
Grade Designation	RS–1	MS–1	MS–2	MS–3	SS–1	SS–2
CHARACTERISTICS						
Viscosity, furol at 77° F, sec	20–100	20–100	100+	....	20–100	20–100
Residue by distillation, per cent	55+	55+	60+	65+	55+	55+
Settlement, 5 days, per cent	−3	−5	−5	....	−3	−3
Demulsibility, per cent						
35 ml $N/50$ $CaCl_2$	60+	....	....	....	....	....
50 ml $N/10$ $CaCl_2$	....	−30	−30	....	−1.0	−1.0
Sieve test, retained on No. 20, per cent	−0.10	−0.10	−0.10	−0.10	−0.10	−0.10
Miscibility with water	....	No appreciable coagulation in 2 hr				
Stone coating	....	No appreciable separation; shall coat the stone thoroughly				
Cement mixing, maximum	....	....	....	....	2.0 per cent emulsion broken	
Residue from distillation						
Penetration at 77° F, 100 g, 5 sec	100–200	100–200	100–200	100–200	100–200	40–90
Solubility in carbon disulphide, per cent						
Petroleum asphalt	97.5+	97.5+	97.5+	97.5+	97.5+	97.5+
Native asphalt	95.0+	95.0+	95.0+	95.0+	95.0+	95.0+
Ash, per cent	−2.0	−2.0	−2.0	−2.0	−2.0	−2.0
Ductility at 77° F	40+	40+	40+	40+	40+	40+

TABLE 7

Specifications for Asphalt Cements

(*American Association of State Highway Officials*)

Penetration Grades	30–40, 40–50, 50–60 60–70, 85–100 100–120 120–150	30–40, 40–50 50–60, 60–70	85–100, 100–120 120–150	30–45 Asphalt Filler
Prepared from	Petroleum	Native Asphalt		Petroleum
General Requirements	Homogeneous free from water, and shall not foam when heated to 175° C			
Characteristics				
Penetration at 25° C, 100 g, 5 sec	See Note 1	See Note 1	See Note 1	30 to 45
Solubility in carbon disulphide, per cent	99 5+	See Note 2	95 0+*	99 0+
Proportion of bitumen soluble in carbon tetrachloride, per cent	99 0+	99 0+	99 0+	99 0+
Ductility at 25° C, cm	30+	30+	30+	4+
Flash point, Cleveland open cup, ° C	175+	175+	175+	
Loss at 163° C, 50 g, 5 hr, per cent	1 0−	3 0−	3 0+	1 0−
Penetration of residue at 25° C, 100 g, 5 sec compared to original penetration before heating	60 0+	50 0+	50 0+	50 0+
Penetration at 0° C, 200 g, 1 min				10+
Penetration at 46° C, 50 g, 5 sec				90−
Softening point, Ring and Ball, ° C				75 to 85

Note 1 Penetration limits for grade desired are to be inserted

Note 2 Limit of 60 0 per cent + for Trinidad asphalt cement and 94 0 per cent + for Bermudez asphalt cement

* Bermudez asphalt cement

Hard Asphalts. Hard asphalts are residual products and have no direct application to road and street uses. Powdered asphalt is hard asphalt which has been finely ground; it is utilized in some road-oil products and in some cold-laid bituminous paving mixtures.

TABLE 8

SPECIFICATIONS FOR SOFT ASPHALT CEMENTS FOR SURFACE TREATMENT

Penetration Grades	150–200	200–250	250–300
CHARACTERISTICS			
Penetration:			
At 0° C, 200 g, 1 min	35+	40+	60+
At 25° C, 100 g, 5 sec	150–200	200–250	250+
At 46° C, 50 g, 5 sec	300+	300+	300+
Solubility in carbon disulphide, per cent	99.5+	99.5+	99.5+
Proportion of bitumen soluble in carbon tetrachloride, per cent	99.0+	99.0+	99.0+
Ductility at 25° C, cm	50+	50+	50+
Flash point, Cleveland open cup, ° C	175+	175+	175+
Softening point, Ring and Ball, ° C	......	......	80 to 115
Loss at 163° C, 50 g, 5 hr, per cent	1.0−	1.0−	1.0−
Penetration of residue at 25° C, 100 g, 5 sec, compared to original penetration before heating	70+	70+	70+

The grades designated as 100–120, and 120–150, penetration in Table 7 are also used for surface treatment.

TAR PRODUCTS

Comparison of Asphalts and Tars. Tars are produced as distillates by the *destructive* distillation of organic material. When tars themselves are subjected to *fractional* distillation they yield pitches. Thus they differ from asphalts, which are residual products of fractional distillation of organic material. Tars also differ from asphalts with respect to various properties, the most important being toxicity, susceptibility to changes in temperature, free carbon, and hardening after incorporation in a wearing surface or pavement.

Tars possess toxicity to a high degree, whereas asphalts do not. This difference makes creosote oils (which are tar distillates) very valuable as preservatives for structural timber. Tar is much more adhesive than asphalt. Tar is much more susceptible to temperature changes than asphalt, that is, it softens more readily when heated and becomes more brittle at low temperatures than asphalt. Some tars contain large percentages of free carbon, whereas asphalts usually contain very small amounts. Gas-house tar generally contains much more free carbon than coke-oven tar, and water-gas tar contains very little free carbon (usually not more than 1 and seldom more than 2 per cent). Tars harden somewhat more rapidly than asphalts in surface applications.

Grades and Uses of Tar Products. Tars frequently are described as light, medium, and heavy with reference to their consistency, but these terms are very general. The penetration test, by which many grades of asphalt may be identified, is not well adapted to semi-solid tars, and the various grades of tar are therefore identified by viscosity and float tests. Table 9 contains recommended grades of tar for various purposes.

Reference to Table 9 shows that tars are not adapted to all the purposes for which asphalts are used. Tars are suitable for surface treatment, as admixtures for soils, for cold-laid bituminous mixtures, and in bituminous macadam pavements. Their greater susceptibility to temperature changes and their more rapid hardening in use has prevented their development as bituminous cements for wearing-course mixtures composed of hot-mixed bituminous concrete or hot-mixed fine-aggregate mixtures.

SHIPMENT AND HANDLING OF BITUMINOUS MATERIALS

Package Shipment. Package shipments of bituminous materials are generally made in steel drums, wooden barrels, or wooden tubs. Liquid materials, such as residual oils, cut-back asphalts, and asphalt emulsions, are shipped in tight steel drums having small outlets fitted with screw-type plugs. Semi-solid products are shipped in comparatively light, closed steel drums, or, occasionally, in wooden barrels. These drums usually are made in two or more weights, and the contents are removed by chopping off the end of the drum or the entire drum.

Bulk Shipment. Bulk shipments of bituminous materials are made in specially constructed tank cars. Tank cars for the shipment of semi-solid products and the heavier liquid products are equipped with steam coils for heating the material for convenient removal; some tank cars are insulated to lessen the cost of heating. The capacity of tank cars varies; 10,000-gal cars are most common, but 8000-gal cars are also used, and 6500-gal cars to a limited extent.

TABLE 9

SPECIFICATIONS FOR TARS

(American Association of State Highway Officials)

Grade Destination	RT-1	RT-2	RT-3	RT-4	RT-5	RT-6	RT-7	RT-8	RT-9	RT-10	RT-11	RT-12	RTCB-5	RTCB-6
Typical Uses*	Prime coat		Prime coat, carpet coat		Carpet coat, road mix		Carpet coat, road-mix, plant mix, seal coat			Carpet coat, plant mix, seal coat, bituminous macadam, crack filler			Carpet coat, road-mix, plant mix, when low-temperature application and quick setting are desired	
CHARACTERISTICS														
Specific gravity, 25°/25° C	1.08+	1.08+	1.09+	1.09+	1.10+	1.10+	1.12+	1.14+	1.14+	1.15+	1.16+	1.16+	1.09+	1.09+
Specific viscosity, Engler														
At 40° C	5–8	8–13	13–22	22–35										
At 50° C					17–26	26–40							17–26	26–40
Float test, seconds														
At 32° C							50–80	80–120	120–200					
At 50° C										75–100	100–150	150–220		
Water, per cent, by volume	2.0–	2.0–	2.0–	2.0–	1.5–	1.5–	1.0–	0	0	0	0	0	1.0–	1.0–
Solubility in carbon disulfide, per cent	88+	88+	88+	88+	83+	83+	78+	78+	78+	75+	75+	75+	80+	80+
Total distillate, per cent, by weight														
To 170° C	7–	7–	7–	5–	5–	5–	3–	1–	1–	1–	1–	1–	1–8	1–8
To 200° C													5+	5+
To 235° C													8–18	8–18
To 270° C	35–	35–	30–	30–	25–	25–	20–	15–	15–	10–	10–	10–		
To 300° C	45–	45–	40–	40–	35–	35–	30–	25–	25–	20–	20–	20–	35–	35–
Softening point of distillation residue														
Ring and Ball, ° C	35–60	35–60	35–60	35–60	35–65	35–65	35–65	35–65	35–65	40–70	40–70	40–70	40–70	40–70
Application temperatures*	60°–125° F		80°–150° F				150°–225° F			175°–250° F			60°–120° F	

* For guidance of user, not a part of the specification

Unloading Tank Cars. Liquid materials are unloaded from tank cars by pumping direct into storage tanks or distributors if the material is to be used immediately upon receipt. Most distributors have pumps which are arranged for unloading of tank cars as well as for spraying the material. Non-inflammable materials are sometimes unloaded by gravity methods.

Heavy viscous liquid products and semi-solid products must be heated for unloading. Ordinary steam boilers often do this heating at stationary or portable paving plants. Portable tank-car heaters, widely used for unloading these materials, are of three types—the condensing type of steam boiler, the superheated oil heater, and the "booster" type of heater. A series of oil burners heat the boiler pipes in the first type, forcing superheated steam through the tank-car steam coils. In the second type a non-volatile oil, heated to high temperatures by oil burners, is forced through the tank-car coils and back to the heater. Heaters of these two types, or other types of steam boilers, are required if the material is semi-solid or an extremely viscous liquid. The "booster" type of heater is used to supplement other car heaters when heavy semi-solid materials must be heated to temperatures which are too high for superheated steam to be economical, and as direct heaters for liquids which are sufficiently viscous to require heating but, at the same time, sufficiently fluid to flow at ordinary temperatures from the tank car to the pump. Medium to heavy cut-back asphalts are examples of such materials. These heaters are equipped with pumps which force the material into pipes heated directly by oil burners; they are extremely rapid in operation but are more costly than steam heaters.

Measurement of Bituminous Materials. Bituminous materials are purchased on the basis of both weight and volume, the ton and gallon being the common units of measurement. All bituminous materials increase in volume upon heating, and it is standard practice to take the volume at 60° F as the basis of volume measurement. The American Society for Testing Materials recommends the following coefficients of expansion for petroleum asphalt products intended for highway purposes:

Gravity Range Degrees, A P I, * at 60° F	Coefficient of Expansion per Degree Fahrenheit
Up to 14 9°	0 00035
15 0° to 34 9°	0 00040

* The American Petroleum Institute gravity scale is based on the following arbitrary relationship.

$$\text{Degrees API} = \frac{141.5}{\text{Sp G } 60°/60° \text{ F}} - 131\ 5$$

Generally, liquid asphalts RC-0, RC-1, MC-0, MC-1, SC-0, and SC-1 have specific gravities from 15.0° to 34.9° API, and the heavier products have specific gravities below 15.0° API. A coefficient of expansion of 0.00037 per degree Fahrenheit is widely used for road tars.

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CHAPTER VII

THEORY OF STRUCTURAL DESIGN OF FLEXIBLE PAVEMENTS

The rational design of roadway cross-sections has received the serious attention of highway engineers since about 1900, when motor vehicles began to appear in large numbers and created a demand for the extensive development of highway and street systems. The problem of design involves the consideration of numerous conditions, of which the most important are the amount and distribution of loads, the supporting capacity of subgrades, the resistance of roadways to wear, the expansion and contraction of roadway surfaces, and the structural strength of roadway materials.

Pavements or roadways were described in Chapter I as being *rigid* or *flexible*. Rigid pavements are those which possess considerable resistance to bending, and flexible pavements are those which possess very little resistance to bending. As these two general classes of pavements exhibit somewhat different behavior under loads they will be discussed separately. Since all the low-type and intermediate-type roadway surfaces (as classified in Chapter I) are flexible types, it will be desirable to discuss the theory underlying the determination of their thickness before describing each individual type. The theory of the structural design of concrete pavements will be described in Chapter XI.

Thickness of Flexible Pavements. The thickness of flexible pavements has been determined almost entirely on the basis of past experience. During recent years, theoretical and experimental studies in the field of soil mechanics have stimulated interest in the development of a rational method of determining thickness. The similarity of the problem of determining the thickness of airport runways also has contributed to this development.

Figure 55(a) represents a cross-section of a flexible pavement having a thickness of d inches and subjected to a concentrated wheel load of W pounds. The load is assumed to be distributed through the pavement to the subgrade in the form of a cone whose surface is at an angle θ with the vertical axis of the cone (the axis being assumed to be at right angles to the surface of the pavement), and the unit pressure of q pounds per square inch on the subgrade is assumed to be uniform. The radius of the base of the cone on the subgrade

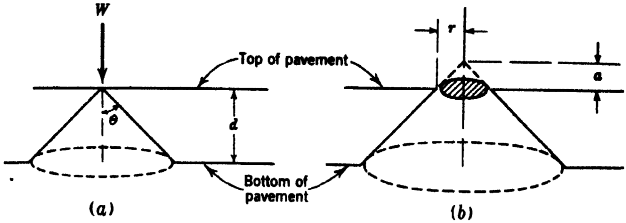

Fig. 55. Distribution of wheel loads through flexible pavements to the subgrade.

is $d \tan \theta$, and the relation of the thickness of the pavement to the load and the unit subgrade pressure is

$$W = q\pi d^2 \tan^2\theta \tag{20}$$

$$d = \frac{0.564}{\tan \theta}\sqrt{\frac{W}{q}} \tag{21}$$

The value of θ varies, but 45° is assumed to be on the side of safety and equation 21 then becomes

$$d = 0.564\sqrt{\frac{W}{q}} \tag{22}$$

Equation 22 was the basis of the Massachusetts formula which was developed in 1901 and was expressed as follows:

$$d = 0.5\sqrt{\frac{W}{q}} \tag{23}$$

Equation 20 assumes that a wheel load is concentrated at a point, whereas it is actually distributed over an area of contact between the tire and the pavement as shown in Fig. 55(*b*). If this area of contact is circular in shape,

$$W = q\pi(d + a)^2 \tan^2 \theta \tag{24}$$

in which a is the height of the cone of pressure above the surface of the pavement; and

$$d = \frac{1}{\tan \theta}\sqrt{\frac{W}{\pi q}} - a \tag{25}$$

or

$$d = \frac{0.564}{\tan \theta}\sqrt{\frac{W}{q}} - a \tag{26}$$

If p is the unit pressure of the load W on the surface of the pavement,

$$W = p\pi r^2$$

and

$$d = \frac{r}{\tan\theta}\sqrt{\frac{p}{q}} - a \tag{27}$$

Since

$$a = \frac{r}{\tan\theta}$$

$$d = \frac{r}{\tan\theta}\left(\sqrt{\frac{p}{q}} - 1\right) \tag{28}$$

If $\theta = 45°$, equation 28 becomes

$$d = r\left(\sqrt{\frac{p}{q}} - 1\right) \tag{29}$$

If the area of contact is elliptical in shape and $\theta = 45°$, the area of pressure on the subgrade also is an ellipse whose major and minor axes have diameters of $2(r_l + d)$ and $2(r_s + d)$, respectively, $2r_l$ and $2r_s$ representing the major and minor axes of the elliptical contact area between the tire and the pavement. Then

$$W = p\pi r_l r_s = q\pi(r_l + d)\,(r_s + d) \tag{30}$$

and

$$d = \sqrt{\left(\frac{p}{q} - 1\right)r_l r_s + \tfrac{1}{4}(r_l + r_s)^2} - \tfrac{1}{2}(r_l + r_s) \tag{31}$$

The application of rational theoretical analyses to the determination of flexible pavement thicknesses is not a simple problem. Several assumptions must be made with reference to (1) the shape and size of the area of tire contact, (2) the critical load, (3) the effect of impact, (4) the distribution of pressure on the subgrade, and (5) the critical condition for failure of the pavement under load.

The area of contact between a loaded pneumatic tire and a plane surface is actually elliptical in shape but the simplicity of calculations is greatly increased if the area is assumed to be circular in shape. The necessity of employing an elliptical contact area is debatable. The size of the tire contact area has generally been assumed to be a function of the load and the inflation pressure of the tire, and to be approximately equal to the load on the tire divided by the inflation pressure of the tire. Thus, a 14 00 by 24-in. balloon tire, inflated to 90 lb per sq in. and carrying a load of 9000 lb would have a contact area of approximately 100 sq in. This assumption neglects the "supporting factor" of the tire which acts to increase the contact area. Under average conditions, the actual area of contact between new truck and bus balloon tires and the pavement may be closely approximated by means of the formula[1]

$$A = (0.75 + 0.03D_t)\frac{W}{p'}, \tag{32}$$

in which A = the area of tire contact, in square inches.

D_t = the nominal diameter of the tire cross-section, in inches.

W = the load on the tire, in pounds.

p' = the inflation pressure of the tire, in pounds per square inch.

The value of p then would be W/A. Thus, the area of tire contact of a 14.00 by 24-in. balloon tire, inflated to 90 lb per sq in. and carrying a load of 9000 lb, would be

$$A = (0.75 + 0.03 \times 14)\frac{9000}{90} = 117 \text{ sq in.}$$

$$p = \frac{9000}{117} = 77 \text{ lb per sq in.}$$

Some designers may desire to be conservative and neglect the tire supporting factor, which will result in slightly larger values of d from equations 29 and 31.

The Uniform Vehicle Code provides for a maximum wheel load of 9000 lb on balloon tires and 8000 lb on high-pressure tires. Single high-pressure tires carrying this maximum load would exceed their recommended load capacities, and, if high-pressure tires were employed, dual tires would be required for proper equipment. A single 14.00 by 24-in. balloon tire is capable of carrying a load of 9150 lb when inflated to 90 lb per sq in. (see Table 10), and therefore might be considered, quite properly, as the critical tire size for the recommended legal maximum load of 9000 lb. The necessity of providing for overloads, undersize tires, and other special conditions is involved in the selection of the critical load condition, and opinion differs widely, as might be expected, in regard to the allowances which should be made for such conditions. The judgment of the designer will have to guide him in a given case. An allowance for impact, usually accomplished by multiplying the static load by an "impact factor," must also be considered. Again opinion differs widely. Impact, of course, exists at the surface of pavements, particularly if a vehicle rides over some object resting on the pavement or if the pavement is irregular in the longitudinal contour of its surface. However, the proportion of the impact which is transmitted through a flexible pavement to the subgrade depends upon the dynamic resistance of the pavement. The time of duration of moving loads is generally very short and tends to make the pavement behave like an elastic solid. This fact tends to support the view of many engineers that no allowance for impact is necessary.

---

[1] Developed by the Tire and Rim Association, Inc., Akron, Ohio.

The distribution of a load on a flexible pavement through the pavement to the subgrade was assumed in the foregoing discussion to be in the form of a cone whose surface is at an angle θ with the vertical axis of the cone, and that the resulting pressure on the subgrade was uniform. Actually, the pressure distribution on the subgrade is not uniform but is in the form of a bell or helmet, the maximum pressure occurring directly under the center of the load, as described in Chapter II.

TABLE 10

RECOMMENDED MAXIMUM SERVICE LOADS AND INFLATION PRESSURES FOR TRUCK AND BUS BALLOON TIRES

(The Tire and Rim Association, Oct. 25, 1940)

Tire Size, inches	Inflation Pressure, lb per sq in.	Load, lb	Tire Size, inches	Inflation Pressure, lb per sq in	Load, lb
6 00-17	50	1250	9 00-15	65	2875
6 00-20	50	1400	9 00-18	65	3225
6 50-17	50	1500	9 00-20	65	3450
6 50-18	50	1575	9 00-22	65	3675
6 50-20	50	1700	9 00 24	65	3925
7 00-17	55	1725	10 00-15	70	3375
7 00-18	55	1800	10 00 18	70	3775
7 00-20	55	1950	10 00 20	70	4000
7 50-15	55	1825	10 00-22	70	4275
7 50-17	55	2000	10 00-24	70	4550
7 50-18	55	2100	11 00-18	70	4200
7 50-20	55	2250	11 00 20	70	4500
7 50-24	55	2550	11 00-22	70	4750
8 25-15	60	2275	11 00 24	70	5000
8 25-18	60	2550	12 00-18	80	5125
8 25-20	60	2750	12 00-20	80	5475
8 25-22	60	2950	12 00 20	80	5800
8 25-24	60	3125	12 00 20	80	6150
			13 00-20	85	6750
			13 00 24	85	7575
			14 00-20	90	8200
			14 00-24	90	9150

✓The practical application of equations 29 and 31 to the determination of thickness therefore depends upon establishing a reasonably accurate relation between the value of q and the maximum value of the subgrade pressure which will cause failure of the pavement. This relation is illustrated in Fig. 56, in which the maximum allowable unit pressure on the subgrade is represented by q_m. Several methods of determining the critical pressure have been suggested. Most of these methods involve the determination of the unit pressure for a critical vertical displacement or deformation of the subgrade. The theoretical vertical pressure at any point within a homogeneous elastically isotropic mass resulting from a load applied at the upper horizontal surface may be calculated from the well-known Boussinesq formula (see Chapter II) or from modifications of the Boussinesq formula based on experimental data. Similarly, formulas have been developed for calculating the deformations of elastic materials under loads. The vertical displacements of soils under loads, however, do not correspond to those of elastic materials. L. A. Palmer and E. S. Barber of the U. S. Public Roads Administration have suggested that the critical deformation be calculated from the results of triaxial compression tests of the soil, by means of the formula

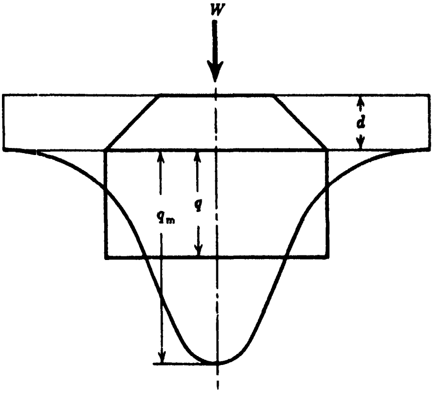

FIG 56 Relation between maximum subgrade pressure and assumed allowable uniform pressure on subgrade. (*A T. Goldbeck*)

$$q = \frac{C_s V}{1.5r}$$

in which V is the allowable vertical displacement of the subgrade, in inches, and C_s is the "modulus of deformation" of the subgrade, which is defined as "the ratio of stress to deformation without regard to the nature of the deformation whether it be elastic or plastic or both." Palmer and Barber's equation for determining the thickness of a flexible pavement subjected to a uniform load on a circular contact area is as follows:

$$d = \frac{r}{\sqrt{C_p/C_s}}\sqrt{\left(\frac{p}{q}\right)^2 - 1}$$

in which d, r, p, q, and C_s have the significance already described and C_p is the modulus of deformation of the pavement. The application of Palmer and Barber's equation will depend upon the accumulation of data for the accurate evaluation of C_p, C_s, and V.

Prevost Hubbard and F. C. Field, in 1940 and 1941, proposed a method of determining the thickness of asphaltic pavements which depends upon the measurement, under field conditions, of the unit pressure which produces a critical vertical deformation of a given subgrade. Hubbard and Field's tests indicated that both asphaltic concrete and sheet asphalt pavements, regardless of the diameter of the area of load application or of the thickness of the pavement, showed no evidence of failure when the vertical deflection of the pavement directly under the load did not exceed 0.5 in. This method possesses the disadvantages that the complete design of a pavement cross-section cannot be provided until the subgrade has been constructed and that, owing to variations in the moisture content of the subgrade after the construction of the pavement, the load-supporting ability of the subgrade may be quite different from that obtained in the actual loading tests. The effect of long-time accumulations of small deformations of the subgrade also is unknown.

✓A. T. Goldbeck, in 1941, proposed a method of determining the thickness of flexible pavements which depends upon the measurement of the load-deformation relation of the subgrade soil. Goldbeck selected a circular bearing block having an area of 100 sq in. and measured the deformations of the subgrade for load increments of 5 lb per sq in. applied for successive 1-min intervals. A pressure-indentation diagram is plotted as shown in Fig. 57. The criterion for the maximum allowable unit pressure on the subgrade is that unit pressure at which the slope of the diagram reaches a value of 100 when the load is expressed in pounds per square inch and the indentation in inches. The slope of the diagram is the ratio of the unit pressure to the indentation and has been designated the "modulus of subgrade reaction." In the example of Fig. 57, the critical unit pressure on the subgrade is 48 lb per sq in. Goldbeck assumed that the maximum unit pressure on the subgrade caused by the wheel load should not exceed the value obtained in the load-indentation tests; thus the value obtained in the load-indentation tests is the pressure designated as q_m in Fig. 56. Goldbeck expressed the relation between this maximum unit pressure and the allowable unit pressure, q, on the subgrade as

$$q = \frac{q_m}{J}$$

in which J is the ratio of the maximum unit pressure to the allowable unit pressure. Goldbeck found that, for stable subgrades, the value of J varied from 1.0 to 3.1 with an average value of 1.93, and suggested a value of $J = 2.0$ for general design purposes. Goldbeck employed elliptical areas of contact between the tire and the pavement, assuming that $r_l = 2r_s$, but his method may be utilized with circular areas.

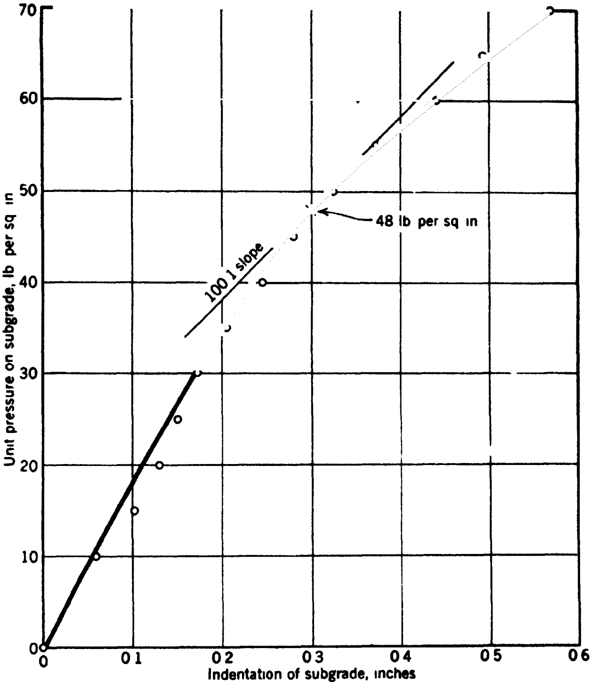

FIG. 57. Pressure-indentation diagram obtained from bearing value test of subgrade. (*A T Goldbeck*)

EXAMPLE. Determine the thickness of a flexible type of pavement for a wheel load of 8200 lb on a single balloon tire. The bearing value of the subgrade was found from tests to be 40 lb per sq in.

W = 8200 lb. Size of tire required = 14.00 by 20 in.

p' = 90 lb per sq in. q_m = 40 lb per sq in.

$$\text{Tire contact area} = (0.75 + 0.03D_t)\frac{W}{p'},$$

$$= (0.75 + 0.03 \times 14)\frac{8200}{90} = 106.6 \text{ sq in. (Say 107 sq in.)}$$

$$p = \frac{8200}{107} = 76.6 \text{ lb per sq in. (Say 77 lb per sq in.)}$$

Radius of tire contact area = 5.84 in.

$$q = \frac{q_m}{J} = \frac{40}{2} = 20 \text{ lb per sq in.}$$

$$d = r\left(\sqrt{\frac{p}{q}} - 1\right)$$

$$= 5.84\left(\sqrt{\frac{77}{20}} - 1\right) = 5.61 \text{ in. (Use 6 in.)}$$

The foregoing discussion of the theory of determining the thickness of flexible pavements and its actual application may appear somewhat disappointing because of the difficulty of accurately evaluating all the variables which affect the problem. The critical factors in the application of the theory are the determination of the tire contact area and the determination of the supporting power of the subgrade. Reported measurements of actual tire contact areas have shown great variations, and some authorities have concluded that the load-inflation-pressure ratio is not a satisfactory criterion for determining the area of contact. From a practical viewpoint, however, it does offer a rational approach to the problem. Certainly, maximum pressures and maximum tire contact areas do exist and the designer always may apply a factor of safety with reference both to the load and to the inflation-pressure if he desires to be conservative. The moisture content and density of the subgrade will greatly affect the measured value of the supporting power. Goldbeck has stated that "The question of condition of the subgrade when the test is made must be left to the judgment of the investigator. It would be desirable, if possible, to bring about the most unfavorable conditions which probably ever will exist under the finished surfacing." It is quite possible that the determination of the supporting power of the subgrade may be determined from laboratory tests of the subgrade soil in which the moisture content and degree of compaction are carefully controlled. This is particularly true of subgrades on embankments where the soil has been disturbed in forming the subgrade. The problem in such cases would seem to be best solved by determining the supporting power of the soil compacted in the laboratory at the optimum moisture content, then following this on the work by compacting the embankment at the optimum moisture content and making such provisions for drainage as will maintain the subgrade at this supporting value. The selection of the size of bearing block for tests of supporting power is subject to great differences of opinion. An area of 100 sq in., which was selected by Goldbeck, corresponds quite closely to the average load-inflation-pressure ratio of the maximum legal load condition for a single tire. The designer, of course, may select any size of bearing area which, in his judgment, appears most appropriate for the conditions involved. Goldbeck's procedure of applying a factor to the maximum measured subgrade pressure to obtain the "equivalent uniform" subgrade pressure is, in effect, the application of a factor of safety to the measured subgrade supporting power. A lower limit on subgrade bearing power is desirable, and most authorities appear to agree that subgrades whose bearing power is less than 20 lb per sq in. should be strengthened by some form of subgrade treatment.

Expansion and Contraction of Flexible Pavements. Flexible pavements and foundation courses are not seriously affected by expansion and contraction, and these movements ordinarily are not considered in design. Flexible wearing courses of bituminous types, when constructed on rigid foundation courses, are affected by the expansion and contraction of the latter, however, and provisions to alleviate detrimental effects will be described later in connection with the subject of rigid foundations.

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CHAPTER VIII

LOW-TYPE ROAD SURFACES AND FOUNDATIONS

The total rural highway mileage of the United States is about 3,100,000 miles. Approximately 350,000 miles may be classified as primary and secondary interstate and state highways. Over 2,000,000 miles of the remaining mileage may be classified as land-service roads and secondary local roads. A large proportion of these 2,000,000 miles is not subjected to sufficient traffic to justify large expenditures for improvement, and the proper utilization of low-cost, low-type wearing surfaces is therefore a very important feature in their development.

Low-type roadway surfaces were described in Chapter I as consisting of natural earth, gravel, traffic-bound macadam. stabilized soil, and special materials which are of localized importance (such as iron ore, shell, and limerock). These surfaces may be untreated, or they may have surface treatments These treatments may consist of surface applications which do not add appreciably to the thickness of the wearing surface, or they may consist of appreciable thicknesses of bituminous materials and mineral aggregate The latter are often described as carpet coats, seal coats, or simply as bituminous surface treatments, and, since they are used on a large variety of wearing courses, they will be described in a later chapter.

NATURAL EARTH SURFACES

Cross-Sections. The width of earth roads extends from shoulder to shoulder, and it is desirable to have as much width as possible so that, when a moderate crown of ½ to ¾ in. per ft is used, the traffic will tend to be distributed over the entire road surface. Widths between shoulder lines are frequently 20 ft on earth roads where shallow gutters can be used, but a minimum width of 24 ft is desirable. Still greater widths are required if surfacing with gravel or other material is planned within a short time after grading.

Construction. The construction of an earth road is essentially a grading operation, and the procedure has been described in Chapter V. The surface of an earth road is usually somewhat loose and rough after shaping, and traffic will cause ruts and depressions. Frequent smoothing with a blade grader or a steel drag, or both, is necessary during the first few weeks, until a firm and smooth surface is maintained.

The construction of natural earth surfaces, without some form of treatment, should not be attempted on many soils. Light sandy soils, heavy clay soils, and many silty soils are unsuitable for wearing surfaces for even very light traffic and require some method of surface stabilization or the construction of a separate wearing course to make them suitable for traffic.

Maintenance. The most effective maintenance of earth roads is accomplished by means of a blade grader in combination with some form of drag. A drag is suitable for smoothing but lacks ability to reshape a road which has become rutted and warped by weather and traffic. Earth roads generally must be reshaped in the spring of the year and after heavy rains, and the motor grader is a very effective machine for this work. Blade graders hauled by tractors, or sometimes by heavy trucks, are also useful for this purpose but require an additional operator in comparison with a motor grader The machines selected for maintaining earth roads also will depend to a large extent upon the total mileage of this type of road under the jurisdiction of a highway organization, upon weather conditions at various times of the year, and upon the character of equipment necessary for other maintenance operations. Since earth roads may be most easily reshaped after rains when the soil is moist, a large amount of equipment might be required for effective earth road maintenance, and the less wealthy organizations often must utilize many types of equipment which may not be completely effective for this one item of maintenance. It is desirable in climates exposed to freezing weather or prolonged rainy seasons to concentrate maintenance on earth roads just prior to these seasons, so that the roads will be affected as little as possible by the adverse weather conditions.

OILED EARTH ROADS

Many clay and silt soils are extremely dusty in dry weather and soften readily in wet weather, and the use of oils to prevent dust and waterproof the surface has been a logical development in many areas, particularly where suitable surfacing materials for light traffic roads are not available locally. The oiling of earth roads is practiced in numerous areas of the United States but has developed to the greatest extent in the Middle West, particularly in the central and southern sections of Illinois where heavy clay and silty loam soils are encountered and where deposits of gravel and stone are scarce.

Oiling is practicable for many soils but is least applicable to soils which consist of almost pure sand and to those which have great capillary rise.

The desirable characteristics of oils for application to earth roads are: (1) they should penetrate and mix with the soil rather than form a surface mat or hard crust, which invariably will break up under traffic; (2) they should not separate into heavy and light fractions and hence be distributed non-uniformly through the soil; and (3) they should waterproof the road surface. For these reasons the residual oils, known as slow-curing liquid asphalt products, are generally more satisfactory than the more rapid-curing cut-back asphalt products. Oils of SC-0 and SC-1 grades, described in Chapter VI, are suitable.

Preparation of the Road Surface. A wide road surface is desirable for an oiled road, particularly on silty soils, in order that traffic will not use a narrow lane but will tend to spread out and prevent rutting. Good drainage is essential, and deep side ditches are often desirable to alleviate the effects of capillarity. The earth surface is carefully shaped and smoothed by blade graders or drags and is pulverized by means of disc harrows or a special heavy discing machine. Blading or dragging after discing is necessary to obtain a smooth and uniform surface. This work is best performed in the spring of the year when the soil is somewhat moist and can be disced and shaped readily.

Oiling. Practice varies in regard to oiling. Silty soils are generally oiled most effectively when the soils are slightly moist, and oiling therefore usually follows immediately after the final dragging or blading of the surface. Oil of Grade SC-0 (Table 4) is most widely used on such soils and usually is applied twice. The first application consists of ¼ to ½ gal per sq yd and may be followed within two or three days by a second application of about ¼ gal per sq yd. Sometimes a third application of about ¼ gal per sq yd is made from one to several weeks after the second application. It is desirable to close the roads to traffic until the oil has penetrated the surface to a sufficient extent to prevent cutting of the surface and splashing on vehicles. In some cases oiling of one-half of the road at a time is practiced to reduce the inconvenience to traffic; a wide road surface is desirable under such conditions.

Clay subgrades which do not have high capillarity are generally allowed to dry out before oiling. They should be smooth and uniform at the time oil is applied and should be trued up when moist and allowed to iron out under traffic. The initial application is of *oil of Grade SC-1 at the rate of ¼ to ⅓ gal per sq yd. After the oil has thoroughly penetrated the surface,* a second application of oil of Grade SC-2 (Table 4) is made, and the oil is covered with a thin layer of ½- to ⅜-in. gravel or coarse sand, which is then bladed or dragged until thoroughly mixed with the oil. This treatment is widely described as the blotter or mulch method of oiling.

Maintenance. The routine maintenance of oiled earth roads consists of blading and patching. The patching often is done with a mixture of the natural soil and oil, or of the blotter covering and oil which is prepared at the time of oiling and stocked in piles or pits along the road.

Re-oiling generally is necessary each year. Oiled earth surfaces often break up in the spring and become very rough; thorough discing and dragging then becomes necessary before re-oiling.

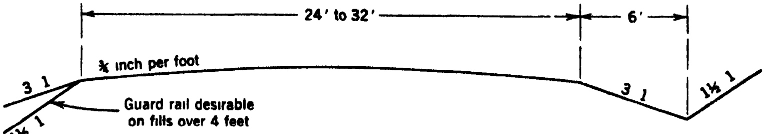
GRAVEL SURFACES AND FOUNDATIONS

The construction of gravel road and street surfaces, when confined to areas in which nearby sources of gravel may be used, is relatively inexpensive because of the low costs of preparing the material, hauling it to the road or street, and constructing the surface.

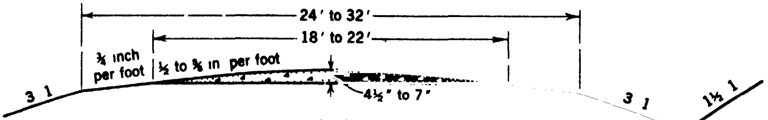
Essential Properties of Gravel. Gravel for road mixtures consists of stone particles or "coarse aggregate," sand, silt, and clay. The dividing line between coarse and fine material varies with different highway organizations and is determined by means of a ¼-in. screen or 4-, 8-, or 10-mesh sieves.

Since the construction of gravel roads is essentially a procedure of utilizing available local materials, wide variations occur in the size, grading, and quality of gravel used. The coarse particles or pebbles should be hard, tough, and durable. The maximum size should not be greater than a 1-in. sieve for a gravel wearing surface but often is restricted to a ¾-in. or ½-in. sieve. Somewhat larger maximum sizes, 1¼-in. and, occasionally, 1½-in., are frequently used for gravel foundation courses. The proportion of coarse material should vary from about 40 to 65 per cent above a No. 4 sieve. The clay content of the mixture should vary from about 7 to 15 per cent. A single material is most desirable, but sometimes coarse and fine material are mixed together on the road.

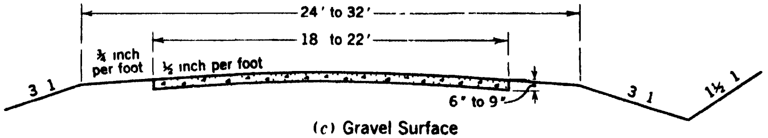
Bank gravels are used most extensively for gravel surfaces and foundations, and the preparation of the material will embrace excavation and screening and may include crushing if the percentage of "oversize" material in the deposit is large enough to justify the use of a crusher. Crushed material is desirable because it adds stability to gravel road mixtures. Screening may consist of the removal of oversize particles only but may also include the removal of part of the fine material. The removal of fine material is avoided wherever possible because the natural moisture content of most gravel deposits causes fine material to clog the screens.



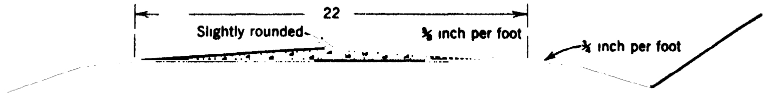
(*a*) Untreated Earth Road



(*b*) Gravel Surface
(Surface method of construction)



(*c*) Gravel Surface
(Trench method of construction)



(*d*) Gravel-type Stabilized Soil Surface

FIG. 58 Cross-sections of low-type road surfaces.

Cross-Sections. Typical cross-sections of gravel roads are shown in Fig. 58. Gravel surfaces are constructed by the "trench" method or by the surface or "featheredge" method. The surface method of construction is less costly than the trench method and possesses the advantage of permitting a road surface to be constructed in a series of relatively thin layers. A somewhat higher content of binder in the gravel is necessary for the surface method of construction because the gravel is not held at the sides by the shoulders and must be firmly held together to prevent being pushed out by traffic. The trench method of construction is applicable to roads on which the gravel layer is to serve as a foundation, and a uniform thickness of gravel is desired.

Construction. *Surface Method.* Subgrades for a gravel surface to be constructed by the surface method are usually prepared by means of blade graders. Subgrades in which clay predominates should be crowned in order to provide drainage and prevent water from collecting under the gravel surface. Sandy subgrades usually are shaped to a flat cross-section because it is advantageous to hold some moisture in the subgrade; the crowned surface then is obtained by having a greater thickness of gravel at the center than is necessary on a crowned subgrade. Subgrades should be smooth and reasonably well compacted before placing gravel, and this is usually accomplished by allowing rain and traffic to consolidate the soil and by smoothing the surface with a blade grader until the subgrade is ready to receive the gravel.

The gravel usually is spread directly on the subgrade by trucks, and care should be taken to secure as uniform a thickness as possible. Thin layers of 2 to 4 in are desirable, and some hand work with rakes and shovels may be necessary to secure a uniform thickness. A blade grader or truck scraper may be used for spreading and smoothing the gravel; a spike-tooth harrow is very good for mixing the material and preventing the formation of a wavy surface. The placing of the gravel in thin layers makes compaction by traffic practicable. The surface should be floated by a blade grader until it is reasonably firm; the next layer then can be placed and compacted in the same manner.

Trench Method. When the trench method of construction is followed, a trench of the required size and shape to receive the gravel is excavated on the subgrade. A blade grader generally is used to excavate the trench to the approximate cross-section when side forms are employed, and the subgrade is finished with a cutting template, which rides on the forms. Sometimes side forms are dispensed with, and the excavated earth from the trench acts as the side support for the gravel layer; this procedure does not produce as true a surface as placing between side forms. The subgrade should be compacted by rolling.

Hauling is generally done by trucks, and the gravel may be spread directly on the subgrade between the side forms. Templates which ride on the side forms may be used for striking off the material to a uniform depth. The maximum depth of loose gravel in one layer should not exceed 6 in., and if the compacted gravel surface is to be more than 4 in. thick, the gravel should be placed in two or more layers. Loose gravel compacts from 25 to 40 per cent, and the extent of compaction must be determined during placing and the thickness of the loose layer adjusted accordingly if the desired compacted thickness is to be obtained.

After spreading, each gravel layer should be harrowed with a spike-tooth harrow in order to obtain a surface of uniform composition and density. Each layer should then be thoroughly compacted. Compaction may be secured by the continued use of a spike-tooth drag alternately with a blade grader or drag, but a tamping roller is a much more satisfactory device. Three-wheel or tandem rollers are not generally satisfactory for compacting gravel layers because they do not compact the material from the bottom of the layer upward but form a hard crust at the top and also generally produce a wavy surface, owing to the pushing of the material in front of the roller wheels. Sheepsfoot and section rollers are most effective. The use of a tamping roller will be most successful when the gravel contains some natural moisture and when rolling is begun immediately after placing and harrowing and is completed before the moisture in the gravel has evaporated. Two or more rollers may be necessary if the gravel is placed in large quantities. The addition of moisture by sprinkling is not generally completely effective; when the practice is followed, the amount of moisture added should be rather small, and the gravel layer should immediately be harrowed and dragged to distribute it as uniformly as possible through the gravel. Sometimes a light three-wheel or tandem roller is used for the final rolling to compact the upper inch or inch and a half of the wearing course.

Calcium Chloride Treatments. Calcium chloride is hygroscopic, that is, it absorbs moisture from the air, and it has been used very extensively for preventing dust and the resulting wear on gravel roads. It is produced in granular and flake forms, the flakes varying from ¼ in. in size to much smaller ones, and the granules being somewhat smaller in diameter. The flake form, which is generally preferred, is usually purchased in moisture-proof paper sacks containing 100 lb, although it also may be obtained in steel drums containing approximately 350 lb. The material is applied dry, direct from the sacks or drums, and the roadway surface should be smooth and firm at the time of application. The presence of an appreciable thickness of loose material on the surface of the road increases the amount of the calcium chloride necessary for binding the surface and therefore should be avoided. Applications are best made with a spreader which is very similar to a lime spreader. Initial applications, following construction, and the first application in the spring, under average conditions, generally should be made at the rate of 1 to 1.5 lb per sq yd, and additional treatments will require about 0.5 lb per sq yd. The number of treatments will vary according to rainfall, traffic, and humidity, as well as the nature of the clay binder and surface condition of the gravel surface; in the Great Lakes region, two to five applications are generally made between the months of May and October.

Calcium chloride treatments are moderately expensive when applied to gravel roads which have been constructed without stabilization of the gravel mixture. Treatment of an 18-ft roadway with 2.5 lb of calcium chloride per year will require about 26,400 lb of material and will cost approximately $300 per mile per year. A large amount of wear of the gravel surface will be prevented, but greater economy may be obtained by the construction of a *stabilized* type of gravel surface; this type of construction will be described later in this chapter.

Courtesy of Solvay Process Co

FIG. 59. Applying calcium chloride to the surface of a gravel road.

Maintenance. The routine maintenance of a non-stabilized type of gravel surface consists of reshaping and "floating" the surface to keep it smooth. Reshaping is usually necessary several times during the year to remove ruts, depressions, and "chatter-bumps." A blade grader hauled by a tractor or a motor grader (see Fig. 80) is very satisfactory for this work, which should be done when the road is moist so that the grader blade will cut easily.

The routine floating of gravel surfaces is performed by means of blade graders, multiple-blade drags, and truck scrapers. The motor grader is more economical than the tractor-hauled grader because it requires only one operator. The multiple-blade drag is a very satisfactory machine for floating long sections of road where its operation is continuous. The truck scraper, which is fastened underneath the frame and between the front and rear wheels of a motor truck, is an efficient and economical machine for routine floating. Speeds of 3 to 8 mi per hr are maintained with truck scrapers. Both straight and curved blades are used on these scrapers, and pressure is applied to the road surface by means of air or oil pressure. The straight blade does not cut as deeply as the curved blade and is suitable for surfaces which are fairly smooth.

Gravel must be added to gravel surfaces at periodical intervals; it is usually applied in layers not exceeding 2½ in. in thickness.

TRAFFIC-BOUND MACADAM SURFACES

Traffic-bound macadam, as its name implies, consists of a broken-stone wearing course which is compacted by traffic, weather, and continuous maintenance. The subgrade is flat or slightly crowned, and the wearing course is constructed by the surface method in much the same way as a gravel wearing course.

Size and Nature of Stone. Limestone is generally used for traffic-bound macadam on account of its cementing property. Crusher-run stone is desirable, and the best results will be obtained with material which has a maximum size of ½ to ¾ in. and of which not more than 25 to 30 per cent will pass through a No. 4 sieve.

Construction. The subgrade is first shaped and compacted, and the stone then is applied in a layer not more than 1 to 1½ in. thick. Traffic is allowed upon the surface immediately, and the surface is bladed at frequent intervals by means of a blade grader. Successive thin layers are then applied as each preceding layer becomes consolidated; frequently the stone is placed at the sides of the subgrade in windrows and is bladed in gradually as the surface compacts.

This type of surface finds its greatest use on local roads in areas where gravel is not available. Its principal disadvantage is the fact that, while being compacted, it is hard on vehicle tires and dusty. When the traffic frequently is over 25 vehicles per day, it is desirable to place a bituminous surface treatment soon after consolidation of the complete surface layer.

Maintenance. The maintenance of an untreated traffic-bound macadam wearing course is similar to that of a gravel road, consisting of periodic blading of the surface and the addition of material where needed to repair breaks or depressions in the surface.

SPECIAL ROAD SURFACES

In many areas of the United States, local materials of a rather special nature are used for low-cost road surfaces and foundations.

Limerock. Limerock is a soft limestone formation consisting principally of carbonate of lime which, when excavated, in many cases is more granular than fragmentary. It has a high natural cementing value which causes the material to bind together. It is found in Florida, Georgia, Alabama, and some other states, but its widest use is in Florida. The construction procedure is very similar to that of a gravel road by the trench method. Side forms are placed, and the material is spread carefully on the subgrade. A thickness of 6 to 8 in. is common, and, in Florida, one-course construction has been very successful. The material is watered and rolled, and is then scarified, watered again, and rerolled. The surface may be given a bituminous surface treatment or may form a base for a bituminous pavement.

Caliche. Caliche is a soft, earthy limestone found principally in Texas and Arizona. Construction methods are very similar to those for Florida limerock.

Low-Grade Iron Ore. Low-grade iron ore, consisting of hematite or limonite, is found in Louisiana, Arkansas, and Texas. It is used in construction similar to gravel and traffic-bound macadam, and, having high natural cementing value, it compacts and binds together to form a firm, smooth surface.

Shells. Shells are used for base and surface courses in Maryland, Florida, Louisiana, and Texas. Both clam shells and oyster shells are used. In Louisiana the former are found in banks in the coastal plain region and the latter in reefs on the Gulf coast. Roads of these materials are built by the trench method; except when subject to light traffic, they should be given a bituminous surface treatment.

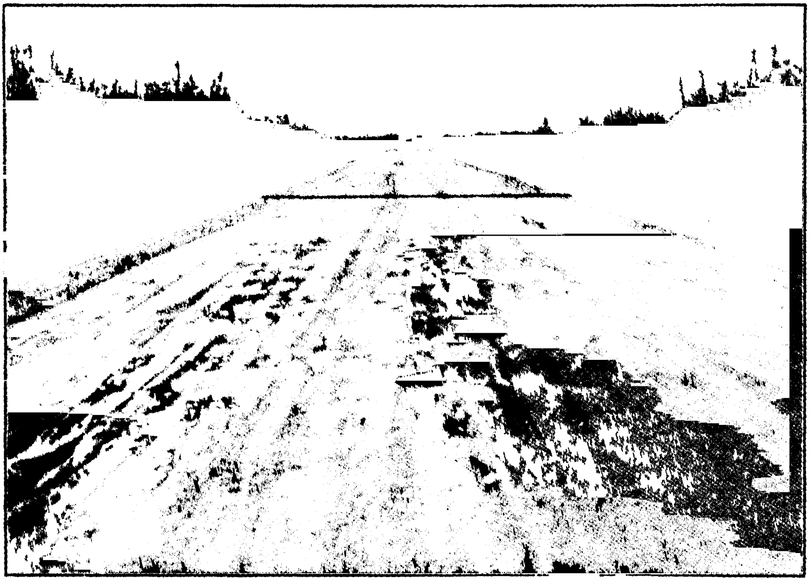
STABILIZED SOIL ROAD SURFACES AND FOUNDATIONS

Our earliest knowledge of stable soil road mixtures was the result of investigations begun in 1906 by Dr C M Strahan in regard to the sand-clay roads of Georgia The theory of sand-clay mixtures was described in the Report of the First Conference (1917) of State Highway Testing Engineers and Chemists, as follows

The present state of knowledge regarding these materials is derived from the comparative study of the road history under traffic of such roads and the mechanical analysis of the constituent materials The data secured indicate the necessity of classification and tests based upon a more complete separation of the ingredients than is usually given in a laboratory analysis Special definitions of the terms sand, silt, and clay are required, also a theory of their mutual interaction after consolidation It is believed that the adhesive clay, acting as a binder in dry weather, and as a stopper to capillary openings by swelling in wet weather, must be supplemented by the interlocking strength of the sand aggregate both in dry and in wet weather, to support the weight and the wear of the traffic, while the non-adhesive silt gives aid to density and water resistance The proportions and sizes of the sand are highly important for the strength and wearing element The clay, owing to its expansion under wet conditions, should be present in minimum amount consistent with sufficient adhesion in dry weather, and the silt should form the inert filler for those voids not occupied by the clay The thorough intermixing of the ingredients is a vital factor, and a process of consolidation which will pack the loose mass from the bottom upward is of much consequence

A further conception of the action of this class of materials suggests that the fine aggregate passing a 10-mesh sieve may be regarded as a mortar capable of being used separately to form the road slab, or of carrying any amount of gravel or coarse aggregate up to the point where a complete gravel road is reached with this mortar as its binder Experience has shown that the addition of coarse material retained on a 10-mesh sieve and less than 2 in in size, provided the coarse material is itself present in graded sizes, increases the durability of these soil mixtures Further the analysis of the fine material passing a 10-mesh sieve in the well-known Augusta, Georgia, gravel, which possesses excellent binding value, reveals a composition similar to some of the best topsoil roads in that state There is reason to think that the binder in washed gravel roads might very well be compounded on the same basis as successful soil road mixtures

The successful use of these road soil mixtures, whether obtained on the surface, as topsoils, from substrata, or from artificial mixtures of clays and sand, depends primarily on the composition as to size, amount, quantity and uniformity of mixing of the three usual ingredients, and upon the adequate consolidation of the mass From the practical standpoint, the selection of the raw material should not be based on a too narrow analytic figure The natural deposits are of variable composition. A number of samples from a selected area close to the work may not conform to a specific percentage. But it is usually possible to supplement deficiencies by adding relatively small quantities of sand or clay, as the case may be, and secure in the mixed roadbed a fairly uniform material whose analysis is satisfactory. The final acceptance of a material of this class should preferably be based on samples taken at specified intervals on the freshly laid and intermixed roadbed and at such other points as the inspector may choose, where the indications are that the mixture is not uniform and satisfactory



Courtesy of Dow Chemical Co

FIG 60 Effect of stabilized soil mixture in preventing softening of road surface in the spring of the year in a cold climate, the road surface in the immediate foreground is of gravel without special treatment—that in the background is a stabilized gravel-type soil surface three years old

The success of the sand-clay roads of the southeastern United States was due in large measure to the excellent binder available in this area Dr Strahan considered kaolin as an ideal binder; its properties, determined by means of the soil tests described in Chapter II, are approximately as follows:

Liquid limit	60
Plasticity index	26
Shrinkage limit	36
Centrifuge moisture equivalent	49
Field moisture equivalent	36

The soil constants of mixtures of clay, sand, and silt are, of course, modified by the sand and silt, and, for highly stable mixtures, will conform to the values given in Table 2 in Chapter II. The diagram of Fig. 16 shows the average grading of stable soil mixtures. A highly stable soil mixture, therefore, should have a binder conforming as closely as possible to the soil constants of kaolin plus a grading of its particles which closely parallels the curve of Fig. 16.

As stated in the preceding quotation, the presence of coarse material (that is, gravel particles) increases the stability of soil mixtures and indicates that gravel road mixtures might be prepared on the basis of adding gravel particles to a stable mixture of sand, silt, and clay. This early conception of the principles underlying successful soil mixtures, together with the additional knowledge of soils provided by more recently developed soil tests, is the basis of methods of constructing stable road and street surfaces of graded soil mixtures.

The impossibility of securing soils possessing all the characteristics necessary for stable mixtures has resulted in the development of other methods and materials for constructing stable soil road surfaces. The use of bituminous materials for soil stabilization has developed rapidly, and Portland cement has proved valuable for stabilization. Thus, stabilized soil surfaces and foundations, at the present time, may be classified into three principal groups: (1) graded soil surfaces and foundations, (2) bituminous types, and (3) foundations stabilized with Portland cement admixtures.

GRADED SOIL SURFACES AND FOUNDATIONS

Graded soil mixtures may be classified as fine-aggregate or sand-clay type of soil mixtures, and coarse-aggregate or gravel-type soil mixtures.

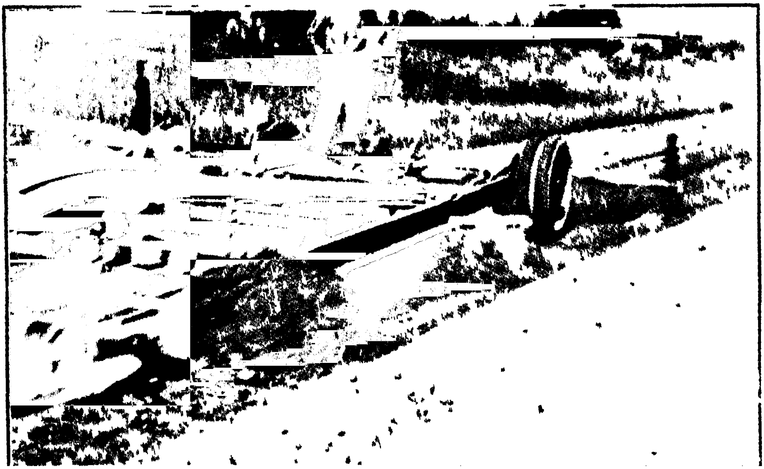
Selection of Materials. The composition of the ideal sand-clay mixture, shown graphically in Fig. 16, and the permissible range in composition of sand-clay mixtures, are as follows:

	Ideal Composition Per Cent	Range in Composition Per cent
Clay	7.5	5 to 10
Silt	15.0	10 to 20
Fine sand (0.05 to 0.50 mm)	25.0	10 to 40
Coarse sand (0.50 to 2.0 mm)	52.5	45 to 60

Gravel-type soil mixtures should be regarded as sand-clay mixtures plus coarse aggregate (that is, material above 2 mm or a No. 10

sieve opening in diameter). The presence of crushed stone, crushed slag, and granulated slag in coarse-aggregate mixtures has been found to increase their stability greatly.

The precise determination of the separate portions of clay, silt, and sand in soil mixtures requires a mechanical analysis by the hydrometer method, which is a lengthy procedure and has not been found necessary. It has become customary to determine the grading of the portion of the soil mixture down to a No. 200 or a No. 270 sieve and rely on the liquid limit and plasticity index of the *soil fines* (that is, the material below a No. 40 sieve opening in size) to control the proportion and quality of the binder. A No. 270 sieve, having an opening of 0.053 mm, which corresponds almost exactly to the dividing line between sand and silt, is somewhat difficult to use; it has been found practicable to substitute the No. 200 sieve. The limitations given in Table 11 are representative of present practice in specifying the composition and characteristics of soil mixtures



Courtesy of Calcium Chloride Assoc

FIG. 61 Mixing gravel-type graded soil mixture by means of a multiple-blade drag

Admixtures. Calcium chloride and sodium chloride (common salt) possess the ability to retard the evaporation of soil moisture and increase the cohesive strength of moisture films on soil particles over that supplied by plain water, and therefore are extensively used as admixtures in graded soil surfaces.

Cross-Sections. Graded soil surfaces are generally constructed by the surface or featheredge method because of the greater adaptability

TABLE 11

Specifications for Soil Mixtures for Wearing Courses and Foundation Courses

Characteristic	Sand-clay Mixtures		Coarse-aggregate Mixtures			
			Wearing Courses		Foundation Courses	
	Wearing Courses	Foundation Courses	1-in maximum size	¾-in maximum size*	2-in. maximum size	1-in. maximum size
Grading						
Passing 2-in sieve	May contain 35% passing 1-in and retained on No 10 sieve; material passing No 10 sieve shall have following grading				100	
Passing 1½-in sieve					70 to 100	
Passing 1-in sieve			100		55 to 85	100
Passing ¾-in sieve			85 to 100	100	50 to 80	70 to 100
Passing ⅜-in sieve			65 to 100		40 to 70	50 to 80
Passing No 4 sieve			55 to 85	70 to 100	30 to 60	35 to 65
Passing No 10 sieve	100		40 to 70	35 to 80	20 to 50	25 to 50
Passing No 20 sieve	55 to 90					
Passing No 40 sieve	35 to 70		25 to 45	25 to 50	10 to 30	15 to 30
Passing No 200 sieve	8 to 25		10 to 25	8 to 25	5 to 15	5 to 15
Fraction passing No 200 sieve as p cent of fraction passing No 40 siev not more than	67		67	67	50	50
Properties of Soil Fines (Fraction Passing No 40 Sieve)						
Plasticity index						
Unusually wet conditions	3 or less	6 or less	3 or less		6 or less	
Average moisture conditions	4 to 8		4 to 8			
Arid conditions	9 to 15		9 to 15			
Liquid limit, not more than	35	25	35		25	

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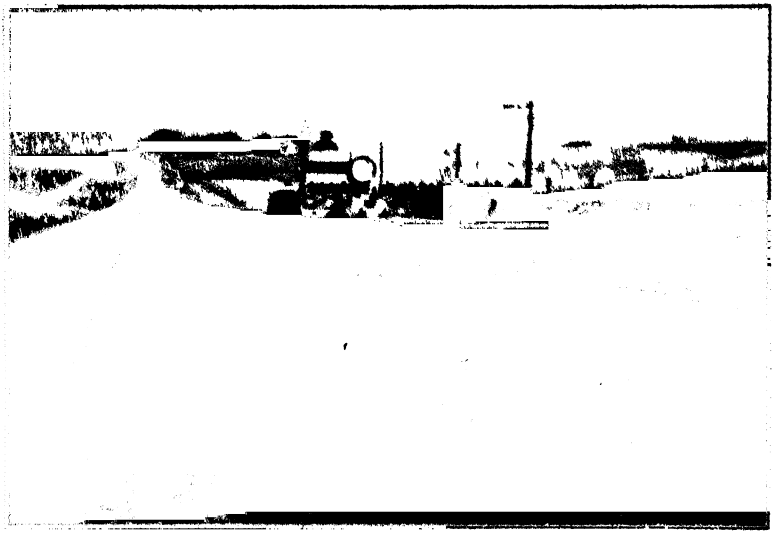
* This grading is suitable for mixtures containing gravel coarse aggregate and should be used for all mixtures containing crushed stone, gravel, or slag.

of this method to road-mix construction. A straight-line crown, with a slightly rounded section at the center of the roadway, is preferred, and the rate of crown, after compaction, should not be less than ⅜ in. per ft; a crown of ½ to ⅝ in. per ft is generally used. Low crowns tend to cause the formation of "potholes." A typical cross-section of a gravel-type, graded soil surface is shown in Fig. 58(*d*).

Construction of Gravel-Type Soil Surfaces. Gravel-type soil surfaces may be constructed by mixing separate materials on the roadway, by using natural soil mixtures having the required composition, and by mixing separate materials prior to placing them on the road. The mixing of separate materials usually consists of mixing a binder soil with a graded soil containing sand and gravel.



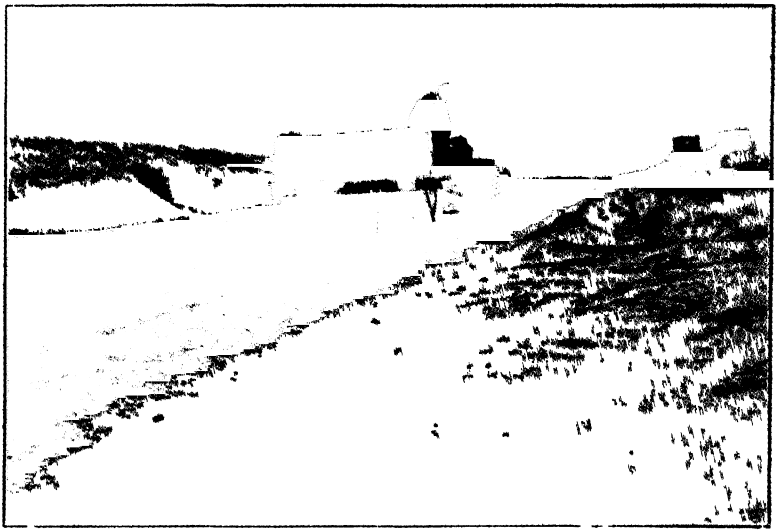
Courtesy of Calcium Chloride Assoc.

FIG. 62 Compacting gravel-type graded soil surface by means of a sheepsfoot roller

Road-Mix Construction. If a binder soil is obtained from the subgrade, it is scarified, broken up with rollers or disc harrows, and bladed (by means of a blade grader) in windrows along the shoulders of the road If all or part of the graded soil is to be obtained from the roadway (as in the case of an existing gravel road), the material in place is scarified and placed in windrows along the shoulders by means of blade graders. The additional material, in either event, then is hauled to the road and distributed along the shoulders, a blade grader being used, if necessary, to place it in windrows.

After the subgrade is firm and hard, and true to cross-section and grade, the materials are thoroughly mixed by alternately spreading and windrowing them with blade graders or by means of multiple-blade drags The mixture then should be placed in windrows on the shoulders.

Soil surfaces of this type may best be compacted by means of rollers when the soil mixture is moist. Rains may be utilized to provide the necessary moisture, or water may be applied by means of sprinklers If sprinkling is practiced, the subgrade is first sprinkled, then about one-fourth of the mixture is bladed over the subgrade and sprinkled, another one-fourth of the mixture is added and sprinkled This procedure is repeated until all the mixture has been spread from the shoulders over the subgrade The sprinkling of the last layer is deferred until the roadway is shaped (by a blade grader) and partially rolled The water then is applied and rolling is continued until the desired degree of compaction is obtained

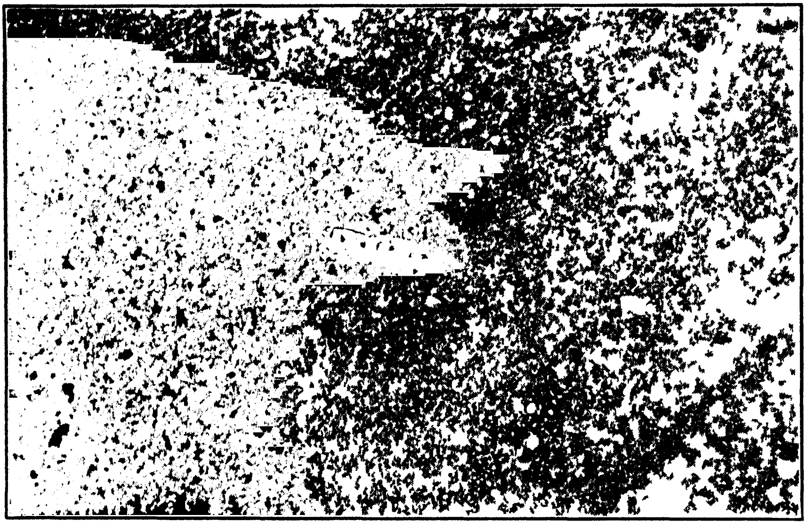


Courtesy of Calcium Chloride Assoc

FIG 63 Compacting a gravel-type graded soil surface by means of a truck-wheel roller

When calcium chloride or sodium chloride is used as an admixture, it generally is added just before the materials are mixed The materials to be mixed are spread loosely over the subgrade and the admixture then is applied and is followed by mixing Calcium chloride is added at the rate of about ½ lb per sq yd per in of compacted surface up to a maximum of 4 in If a surface of more than 4 in in thickness is built, the calcium chloride is added to the upper 4 in only Sodium chloride also is added at about this same rate

Compaction of these surfaces sometimes is left to traffic, but rolling is desirable Three-wheel rollers, tamping rollers, and rollers consisting of a number of pneumatic truck tires, mounted on separate axles, are used.



Courtesy of Calcium Chloride Assoc

FIG 64 Close-up view of a compacted gravel-type graded soil road surface

Frequent smoothing of the surface generally will be necessary for one to three weeks after the road or street is opened to traffic, and additional sprinkling may be desirable

When complete soil mixtures are obtainable from gravel deposits, the procedure of building the stabilized soil surface does not differ from that just described, except that the mixing of the materials, a graded soil and a binder soil is not required

Calcium chloride and sodium chloride are not always incorporated as admixtures, and it may not be convenient, when a single soil of the desired composition is available, to mix the salt with the soil at the source of supply The salt could, of course, be mixed with the soil on the road, but, since the complete soil mixture is already available and merely requires placing, spreading, compacting, and smoothing, the salt is applied to the surface after compaction The same quantity is used as in the former case, up to a maximum of about 2 lb per sq yd.

Plant-Mixed Materials. Adjacent gravel and clay deposits, containing suitable graded aggregate and binder soil, respectively, are found in some localities, and, when the quantity of material to be used is large, may be mixed together at the gravel deposit which furnishes most of the mixture. The entire operation may consist of screening or of screening and crushing the gravel, as well as mixing the two materials, and may be performed at a commercial gravel plant or by means of a portable plant for individual projects. The equipment for plant-mixing of soils includes crushers for pulverizing the clay, proportioning devices, and a mixer. Both wet and dry mixing processes are used. Calcium chloride or sodium chloride usually is incorporated in plant mixtures because of the ease with which they may be handled. The plant-mixed material is hauled to the road or street and spread to the desired shape and thickness. It then is compacted and smoothed by the procedure which already has been described.

Construction by Special Machines. Soil road surfaces have been constructed by means of specially designed machines which pick up the materials from the roadbed, mix them, add water during mixing, and spread the mixture on the roadbed. Such machines are adaptable only when a large amount of work is involved.

Construction of Sand-Clay Type Soil Surfaces. Sand-clay or soil mortar surfaces are adaptable to locations where coarse aggregates are not economically available The construction procedure does not vary greatly from that described for gravel-type surfaces The natural soil in the subgrade may be used for one ingredient if it has the essential characteristics. Sometimes natural topsoils, which fulfill all requirements, are available; in other cases, two or more separate soils may be mixed.

Use of Graded Soil Mixtures for Shoulders. Graded soil mixtures are very desirable for the construction of shoulders on heavily traveled paved roads because they furnish a firm support for vehicles which, necessarily or accidentally, get off the pavement. Plant-mixed materials are desirable:

Graded Soil Foundations. Graded soil mixtures provide very satisfactory base or foundation courses for intermediate- or high-type wearing courses and, therefore, are well adapted to stage construction.

Maintenance of Graded Soil Surfaces. The maintenance of a graded soil surface involves blading or floating, patching, and treatment with calcium chloride or common salt.

The smoothing or floating of an ordinary gravel road is necessarily a practically continuous operation because the surface layer of gravel usually is somewhat porous and quite easily raveled and dislodged by traffic. The surface of an effectively stabilized soil wearing course, on the other hand, is hard and firm and cannot be easily cut by a grader blade when dry. Blading therefore should be done when the surface has been softened by rain. The motor grader (see Fig. 80) is an ideal machine for blading graded soil surfaces. These surfaces may get rough during prolonged periods of dry weather, and sprinkling may be desirable to soften the surface sufficiently for smoothing by means of a blade grader.

Breaks in the road surface or depressions which may form may be repaired by patching, and rough areas may be corrected by patching instead of by means of a blade grader. Patching mixtures should have a maximum size of ½ in. or less when thin patches are to be placed, and calcium chloride or common salt (depending on which was used for construction) should be incorporated in the mixture at the rate of 100 to 175 lb per cu yd to facilitate compaction.

Soil road surfaces are best maintained when treated more or less regularly with calcium chloride or common salt. Two or three treatments per year are generally sufficient. The first application in the spring usually will require about 1 lb per sq yd, and later treatments will require about half of this quantity.

Additional material will be required at periodic intervals (of three to seven or eight years) to replace material lost by the action of traffic, maintenance, operations, and weather. Plant-mixed materials are desirable, but additional thickness may be secured by road-mixing.

Graded soil surfaces are adaptable to bituminous surface treatment if the subgrade beneath them is stable. Surfaces of considerable thickness may be required on capillary soils to develop sufficient support for bituminous surface treatments or thin wearing courses.

BITUMINOUS TYPES OF STABILIZED SOIL FOUNDATIONS

The use of graded soil mixtures is a very satisfactory solution of the problem of securing the necessary support for wearing courses or of the problem of providing relatively inexpensive, low-type wearing surfaces, but it necessarily is limited to areas in which soils having both satisfactory gradings and satisfactory binders are available. The lack of such soils in many areas led to the use of bituminous materials for stabilizing inferior soils. The utilization of bituminous materials as admixtures is comparatively recent, and standardized procedures have not been developed. However, the fundamental principles of soil stabilization by means of bituminous materials are well understood, and considerable success has been obtained in numerous locations.

Bituminous materials may serve a dual function. They may waterproof soils which have detrimental capillarity, and they may provide cohesion for non-cohesive soils. Sandy soils do not require waterproofing, but it is desirable to increase their stability by furnishing cohesion. Cohesive clay soils have great stability as long as their moisture contents are below their plastic limits but are very unstable when their moisture content is equal to, or greater than, their plastic limits; waterproofing clays, therefore, is important in order to maintain their moisture contents low enough for stability. Soils in which silt predominates not only lack cohesion but also are highly capillary. Therefore, the function which bituminous materials must perform exerts a very important influence on the properties which the bituminous materials should possess. Heavy, adhesive materials are necessary to provide cohesion, whereas relatively thin materials may serve satisfactorily for waterproofing.

Soils which are stabilized by means of bituminous materials, unlike graded soil mixtures of the sand-clay or gravel type, are not used for wearing courses for light to moderate traffic without bituminous surface treatment.

The fact that some soils have a greater attraction for air or water films than for the films of bituminous materials used as admixtures was stated in Chapter II, and it may be desirable to add other materials, such as soaps, pure limestone dust, or hydrated lime, in order to obtain the desired adhesion of the bituminous material. The technique of this procedure has by no means been fully developed, but the swell test and the bitumen-water preferential test, described in Chapter II, are useful in determining these characteristics of soils, and the effect of additions of various materials in increasing the adhesive strength of the bituminous film.

Bituminous Materials Used. Bituminous materials which have been used for soil stabilization include light to heavy tars; slow-curing, medium-curing, and rapid-curing liquid asphalt products; and emulsified asphalt.

Light tars (Grades RT-1 to RT-4, Table 9, Chapter VI) are recommended for heavy clay soils, and heavy tars (Grades RT-5 to RT-7) are recommended for predominately sandy soils. Medium to heavy tars are adaptable to soils in which silt predominates.

Slow-curing and medium-curing liquid asphalt products (see Tables 4 and 5, Chapter VI) have given better results than rapid-curing products on account of the separation of the heavy and light constituents of the latter when dependence is placed on diffusion to distribute the bituminous material through the soil.

Emulsified asphalt mixes with and diffuses through soils very readily. Asphalt emulsions must be of the slow-breaking type in order to coat the soil particles thoroughly.

Construction Procedure. The soil to be stabilized must be thoroughly loosened and pulverized by means of scarifiers and disc harrows. The prepared soil then is bladed into windrows at the sides of the roadway and spread over the roadway in layers from 1 to 2 in. in thickness, and the bituminous material is applied to each layer of soil. After all the bituminous material has been applied to the last layer of soil, the whole mass is thoroughly mixed by means of harrows, blade graders, or multiple-blade drags. Some soils require the addition of water in order to facilitate mixing; this is particularly true of clay soils. In some cases the amount of water added has been determined by obtaining the optimum moisture by means of the Proctor test and allowing for the quantity of bituminous material used.

Special mixing machines, which pick up the soil from the roadway, mix it with the bituminous material, and spread the mixture on the roadway, have been used in place of graders and harrows.

Compaction of the mixture is best secured by means of tamping rollers of the types which already have been described. Three-wheel and tandem rollers are used for final compaction of the surface layer.

Final Treatments. Since the mixtures of soil and bituminous material are not designed to resist the abrasive action of traffic, a bituminous surface treatment usually is applied soon after final compaction if a low-type wearing surface is the object sought. If the stabilized soil is to be the subgrade or foundation for a thick wearing course or pavement, the surface treatment is omitted unless a rather long period of time will elapse before the construction of the wearing surface.

Adaptability of Bituminous-Bound Soils. The stabilization of soils by means of bituminous materials has been practiced almost wholly for the purpose of obtaining low-cost road surfaces for light to moderate traffic, but the adaptability of this procedure to securing stable subgrades for more costly wearing courses or pavements may be entirely practicable in many locations where defective subgrades are encountered.

The thickness of bituminous soil mixtures usually has varied from 4 to 6 in. for low-cost road surfaces. Under unusually unfavorable conditions, it would appear necessary to have greater thicknesses for either low-type surfaces or subgrades below intermediate- or high-type pavements.

STABILIZATION WITH PORTLAND CEMENT

Experiments with soil-cement mixtures for light-traffic roads and for stabilizing subgrades under pavements were first undertaken in South Carolina in 1932. The Portland Cement Association, in 1935, initiated an investigation to determine the effect of cement on subgrade soils and to develop effective methods of construction. An extensive progress report of this investigation was published in 1938 and was followed by a report, in 1940, on the results of tests of a large number of soil-cement mixtures. This discussion is based on these reports.

Tests of Soil-Cement Mixtures. Soil-cement mixtures, to be effective, must be mixed with sufficient cement to be durable; that is, they must resist the detrimental effects of moisture and frost when exposed to weather conditions. In addition they must contain that amount of water which will enable them to be compacted, while damp, to a dense mass. Three types of tests are employed to determine the durability and the moisture-density relation: the Proctor compaction test (described in Chapter II), a wetting and drying test, and a freezing and thawing test.

Several mixtures of soil and cement, containing varying proportions of cement (usually 4, 6, 8, and 10 per cent, by weight, of the oven-dry weight of the soil) are prepared, and the optimum moisture content of each is determined by means of the Proctor compaction test.

Specimens of each mixture, mixed to optimum moisture content, are molded and allowed to hydrate in a damp atmosphere for 7 days. One group of specimens is then subjected to alternate wetting and drying and another group to alternate freezing and thawing. The soil losses and the volume and moisture changes in twelve cycles of each test are employed as a measure of the proportion of cement required to contribute satisfactory durability.

Classes of Soils Adaptable to Treatment. Sandy soils (U. S. P. R. A. Groups A-2, A-3, and A-5) and silty soils (U. S. P. R. A. Groups A-4 and A-7) are definitely adaptable to treatment when the following conditions exist:

1. *The liquid limit is less than 50.*
2. *The plasticity index is less than 25.*
3. *The clay content is less than 35 per cent.*
4. The percentage of solids at the optimum moisture content is not less than 60.
5. The soil will develop a characteristically normal moisture-density curve.

The sandy soils in general require about 6 per cent of cement for stabilization and the silty soils from 8 to 10 per cent. Clayey soils (U. S. P. R. A. Groups A-5, A-6, and A-7) may be greatly improved by the addition of about 10 per cent of cement but, in general, do not respond as well as the sandy and silty soils.

Construction Procedure. The construction of soil-cement surfaces or subgrades, after the work of excavation and embankment is completed, involves scarifying and pulverizing of the soil, which are followed by processing. The term "processing" is applied collectively to the operations of adding and mixing the cement and water, compacting the mixture, and final shaping of the surface. The surface is protected for a period of time and may be given a bituminous cover before it is opened to traffic.

Scarifying and Pulverizing. The subgrade or roadway is scarified to a depth generally not over 6 in. and to a width about 12 in. greater than the width of the riding surface by means of road rooters or a scarifier attachment on a road grader. The soil then is pulverized by means of offset disc harrows (10 ft long and having discs not less than 24 in. in diameter). Most soils pulverize readily, but some soils may require drying for a day or two and others may require moistening. A length of 1500 ft is generally pulverized before processing is commenced. The pulverized soil is smoothed to crown and grade by means of a blade grader and, if the work is performed during rainy weather, is covered with canvas or waterproof paper to shed rain. Normally, windrowing of the pulverized soil will be sufficient protection against rain if provision is made for rapidly draining rainwater from the subgrade.

Spotting and Spreading Cement. Sacks of cement are spotted by hand, longitudinally and transversely, at correct spacings to give the required quantity of cement per square yard of surface, which must be calculated in advance. The cement is then dumped from the sacks and spread by hand rakes or by a spike-tooth harrow.

A protective cover, consisting of lumber platforms or about 6 in. of earth, is placed on any adjoining section of completed surface for a distance of 30 to 40 ft, in order to prevent the processing equipment from marring the surface.

Mixing Cement and Soil. The cement and soil are mixed together for the required width and depth (usually 6 in.) by means of spring-tooth heavy-duty field cultivators hauled by small tractors, making repeated trips until a thorough mixture is obtained. Occasional trips of the offset disc harrows may be desirable. A two or three bottom gang plow will speed up the mixing by turning up the bottom portion of the soil-cement mixture which is not mixed so effectively as the top portion by the field cultivators.

Adding and Mixing the Water. The water required for optimum moisture content of the mixture is applied by means of pressure distributors (see Chapter IX) in several increments. Each increment of water is mixed with the soil-cement mixture by means of the field cultivators. When about half the required amount of water has been applied, the entire material is turned over with the gang plows and mixing continued with the cultivators The moisture content of the mixture is checked before the final increment of water is added. The material adjacent to previous work is moved away from the end of the completed section by means of a blade grader and as the final damp mixing is completed is moved back into position. The header board at the end of previous work is removed in this operation.

The moisture content of the soil, before water is applied for mixing, is generally so low that it has but little influence on the hydration of the cement, but the hydration of the cement proceeds normally after applying water and the damp mixing should be completed within 2½ hours.

Initial Compaction and Shaping. The damp soil-cement mixture is compacted by rolling with a sheepsfoot roller, which packs the mixture from the bottom up. When about one-half of the mixture is compacted, a blade grader works with the sheepsfoot roller shaping the roadway to crown and grade, until about 1 in. of material remains to be compacted. A final round trip of the grader is then made, and a spike-tooth harrow makes one or two round trips to remove compaction planes resulting from the operation of the sheepsfoot roller and the grader.

Final Compaction and Shaping. Final compaction is obtained with a tandem roller, rolling first at the edges and gradually moving in to the center of the roadway. After the rolling, the surface sometimes is lightly planed with the blade grader and again rolled in the same manner. The damp mixing and compaction of the surface should be completed within six hours so as not to interfere with the normal setting of the cement. Care should be taken to connect old and fresh work at the same grade.

Protection of Surface. It is essential that the moisture be retained in the compacted surface to ensure the proper hydration of the cement. The surface of the roadway, immediately after final shaping, is covered with 2 in. of shoulder soil or with 4 in. of hay or straw, and this cover is kept continuously moist for seven days, only enough water being applied to wet the surface without causing free water to stand on it. The cover may be removed after seven days and the surface opened to traffic.

Seal Coat or Carpet Coat. When the surface is to serve as a light-traffic roadway, a bituminous seal coat or carpet coat is generally applied, utilizing a mineral aggregate having a maximum size of ½ or ⅜ in. This type of surface will be described in Chapter X.

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CHAPTER IX

BROKEN-STONE SURFACES AND FOUNDATIONS

Modern broken-stone roads first were constructed by Tresaguet (1764) in France, and by Telford (1805) and Macadam (1815) in England. Macadam used material of the same size for the entire thickness of the stone layer and insisted on a maximum size rarely exceeding 1 in. Both Tresaguet and Telford used large pieces of stone in the foundation course and covered them with smaller particles. The term "macadam" has come to mean surfaces constructed of angular fragments of broken stone. Telford's name has been perpetuated in connection with foundations made of large-sized, flat pieces of stone.

Waterbound macadam is a term denoting broken-stone surfaces and foundation courses bound together with a mixture of stone dust and water. The compaction of waterbound macadam layers is secured by rolling with a heavy power roller.

Traffic-bound macadam denotes broken-stone surfaces gradually compacted and bound together by the action of traffic, dragging, and rains. This type of surface was described in Chapter VIII.

Bituminous macadam is a broken-stone layer bound together by means of a bituminous cement which is applied to the surface of the compacted stone layer and penetrates into the void spaces between the stone fragments.

Cement-bound macadam is similar in the method of construction to bituminous macadam, being bound together with Portland cement mortar instead of bituminous material.

WATERBOUND MACADAM SURFACES AND FOUNDATIONS

Waterbound macadam surfaces were well adapted to horse-drawn steel-tired vehicles because the action of the horses' shoes and the steel tires produced fine material to replace that worn away by traffic. They cannot withstand the suction and abrasive action of rapidly moving rubber tires, and they ravel and deteriorate rapidly unless protected by a bituminous wearing surface. Strictly speaking, there-

fore, waterbound layers are seldom used as surfaces but are extensively used as foundations or base courses.

Materials. Crushed limestone and dolomite are used most extensively for building waterbound macadam surfaces and foundations because they possess the essential properties of hardness and toughness and because the fine particles that are obtained in crushing, when mixed with water, form a natural cement. Crushed blast-furnace slag, crushed traprock, crushed granite, and crushed boulders also are used.

Essential Properties of the Stone. The stone should be structurally sound, reasonably hard and tough, and the dust resulting from crushing, when mixed with water, should have cementing properties. Many deposits of comparatively soft stone have been successfully used for waterbound macadam construction, particularly on roads and streets which do not carry large numbers of heavy vehicles. As the weights of vehicles and the proportion of heavy vehicles increase, the higher should be the quality of the stone. A maximum percentage of wear of 6 to 8 in the Deval abrasion test[1] is generally recommended for crushed stone. A unit weight of 60 to 65 lb per cu ft is widely used to control the quality of blast-furnace slag. The Los Angeles abrasion testing machine[1] is rapidly replacing the Deval abrasion machine as a means of determining the quality of broken stone; a percentage of wear of 37 to 40 after 500 revolutions is quite representative of specifications which specify this test for broken stone for waterbound macadam. No generally accepted test for determining the cementing property of stone dust has been developed.

Size and Grading. The same size of crushed stone or crushed slag is generally used in all courses or layers of waterbound macadam surfaces because of the convenience of storing and handling. Soft stone should be larger than hard stone because it fractures to a greater extent when rolled. The American Association of State Highway Officials recommends two sizes of crushed stone as follows:

	Per Cent
1 to 2 in.	
Passing 2½-in. sieve	100
Passing 2- in. sieve	90 to 100
Passing 1½-in. sieve	35 to 70
Passing 1- in. sieve	0 to 15
2 to 3 in.	
Passing 3½-in. sieve	100
Passing 3- in. sieve	90 to 100
Passing 2- in. sieve	0 to 15

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[1] See current *Book of A. S. T. M. Standards* for a description of this test.

A combination of the foregoing two sizes also is used. Larger sizes of stone are sometimes specified, but they are extremely difficult to spread so that a true surface is obtained. The 2- to 3-in. size presents some difficulties in this respect and requires more care in placing than the 1- to 2-in. size.

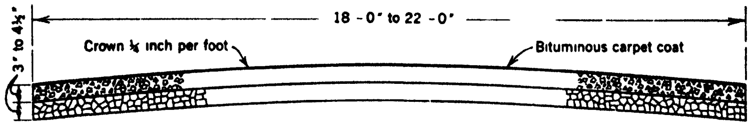
The specifications of the American Association of State Highway Officials provide for two sizes of screenings for filling the voids in waterbound macadam, as follows:

	Per Cent
0 to No. 4 sieve	
Passing ⅜-in. sieve	100
Passing No. 4 sieve	85 to 100
Passing No. 100 sieve	10 to 30
0 to ½-in.	
Passing ¾-in. sieve	100
Passing ½-in. sieve	90 to 100
Passing No. 100 sieve	10 to 30

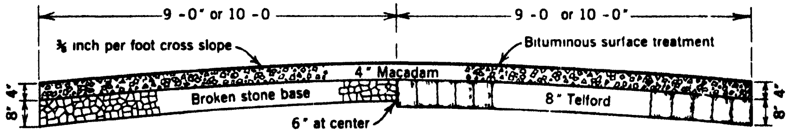
Cross-Sections. The thickness of a broken-stone layer is limited to the depth which may be thoroughly compacted by a heavy roller. This depth is about 6 in., loose. Broken stone compresses under rolling, as a general average, approximately 20 per cent of its loose thickness. If a standard dressed 2-by-6-in. board is used for a side form, the thickness of the loose layer after spreading will be about 5⅝ in. and will compress under rolling to about 4½ in.; this is about the maximum practicable thickness. Two courses or layers of broken stone usually are necessary on ordinary subgrades to secure a suitable thickness; when thicknesses greater than 9 in. are to be provided, the stone should be placed in three layers.

Broken-stone surfaces commonly vary in thickness from 7 to 9 in., although they may be either thicker or thinner, and the thickness may be uniform or variable. With unusually stable subgrades or stable foundations of other materials, a single layer of broken stone may be sufficient. The crown on waterbound macadam roads is somewhat flat because such roads are almost always covered with a bituminous surface treatment. The usual rate of crown is ¼ in. per ft, which gives a total crown of 2½ in. for a width of surface of 20 ft. This rate can be reduced slightly if care is taken to obtain a true surface during construction and to maintain the surface contour afterwards Typical cross-sections are shown in Fig. 65.

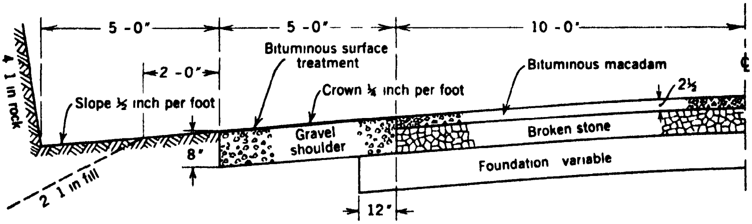
Preparation of Subgrade. The subgrade should be compacted as uniformly as possible. The preparation of the subgrade is especially important because a waterbound macadam layer not only is flexible but it cannot be reshaped, as can a gravel layer, by means of a blade grader or similar machine. Some method of stabilization may be necessary on some soils, particularly sands and silts. Construction on many clay soils is feasible if permeable soils are brought in for the shoulders. Highly capillary and elastic clay subgrades are very unsatisfactory and should be stabilized Construction on sandy soils is very difficult unless clay is mixed with the soil or a blanket course of sand-clay or sand-clay gravel is placed between the subgrade and the first course of stone



Waterbound Macadam Surface with Bituminous Surface Treatment
Two Course Uniform Thickness



Bituminous Surface Treatment Macadam Pavement
Pennsylvania Department of Highways



Bituminous Macadam Pavement
Division of Roads and Bridges, Rhode Island Dept of Public Works

Fig 65 Cross-sections of broken-stone pavements

Waterbound macadam layers are almost always constructed by trenching the subgrade to obtain the necessary quantity of earth for the shoulders. The earth shoulders are often used to confine the edges of the broken-stone layers, but much more satisfactory results will be secured by having side forms for each layer, which are kept in place until the layer has been rolled and bound The shoulders should be shaped and compacted adjacent to the edges of the lower course of broken stone before the stone for the upper course is spread in order to give side support.

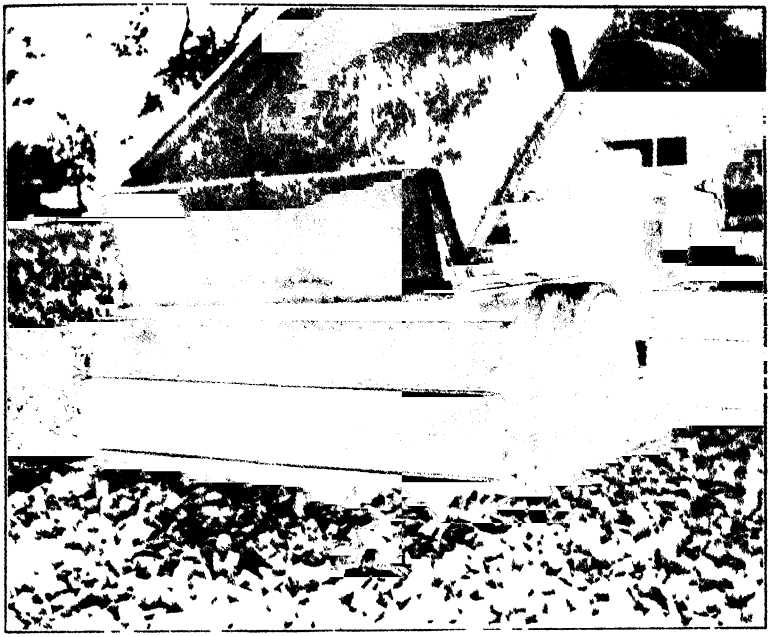
Unloading and Hauling the Stone. *If the stone is obtained direct from a crushing and screening plant, it is loaded direct into the hauling vehicles from bins.* When stone is shipped in cars from producing plants, several methods of loading vehicles may be followed. On work of large magnitude, unloading by means of cranes is economical. On overage work, car unloaders of the belt conveyor type having a hopper placed beneath a bottom-dump (gondola) car are very common.

Broken stone may be hauled in wagons, on industrial railways, or in motor trucks, the last being by far the commonest. Large trucks, having capacities of 3 to 5 cu yd, are desirable for hauling stone.

Placing the Stone. The placing of the broken stone is an exceedingly important part of the construction of a waterbound macadam layer. It is essential that the stone be spread uniformly in order to obtain a surface which will have smooth riding qualities. Dumping the stone in large piles on the subgrade and spreading it out from these piles usually results in the segregation of sizes; the stone which is left in the original pile is more compact than that in the surrounding areas, and uniform compression by means of rolling is practically impossible. Before the development of the present types of dump trucks, it was generally considered necessary to dump the stone on dumping boards or platforms or in piles along the sides of the roadway and to place the stone by means of hand shovels. This method is costly and has been supplanted very largely by the use of mechanical spreaders. It is possible for a skilled truck driver and an experienced helper to spread stone directly from a dump body by carefully controlling the tail gate opening and the movement of the truck; the mechanical spreader, however, provides a more efficient and accurate method.

In order to secure the proper and uniform thickness of a course of broken stone, side boards, or forms, of correct height are necessary. Center boards sometimes are used, but rows of stakes set at proper elevations or of blocks equal in height to the correct depth of the loose layer are more satisfactory. String grades also are used for this purpose. The necessity of properly preparing the subgrade and of having its surface parallel to the finished surface is apparent. A check on the placing of the stone may be secured by calculating the longitudinal distance which each load of stone should cover.

Rolling the Stone. After the stone has been placed, it is rolled with a heavy roller which should be of the three-wheel, or macadam, type and weigh not less than 10 tons. Rolling should begin at the sides of the broken-stone layer if side forms are used and should overlap the shoulders for a distance of 12 to 18 in. if they are not. It should progress gradually toward the center, parallel to the longitudinal axis of the road, with a uniform overlap on each succeeding trip of the roller. Rolling is continued until the stone has become thoroughly keyed; this condition is reached when the stone no longer creeps or waves ahead of the roller During the first trips of the roller, low places may develop in the stone layer, they should be corrected by slightly roughening the surface and adding more stone



Courtesy of the Burch Corporation

Fig 66 Mechanical spreader for placing broken stone The box is pulled by the truck

Application of Screenings. The screenings may be spread by means of mechanical spreaders or by hand When hand methods are used, the screenings should be deposited in piles along the roadway or on dumping boards and broadcast over the surface with shovels with a circular sweeping motion to a depth just sufficient to cover the stone. The surface is now broomed with fiber brooms which push the screenings into the void spaces in the stone. The surface is then thoroughly rolled and more screenings added in small quantities until no more screenings can be forced into the voids in the coarse stone.

The broken-stone layer then is sprinkled with water and more screenings added when necessary. The sprinkling, adding screenings, sweeping, and rolling is known as *puddling* and is a very important part of the construction of a macadam layer. The puddling operation is continued until a wave of grout flushes ahead of the roller. The quantity of screenings required to fill a waterbound macadam layer will vary according to the size and quality of the coarse stone, the size of the screenings, and the amount of puddling, and generally will be equal to about 20 per cent of the volume of the compacted layer for base and intermediate courses, and about 25 per cent for the top course.

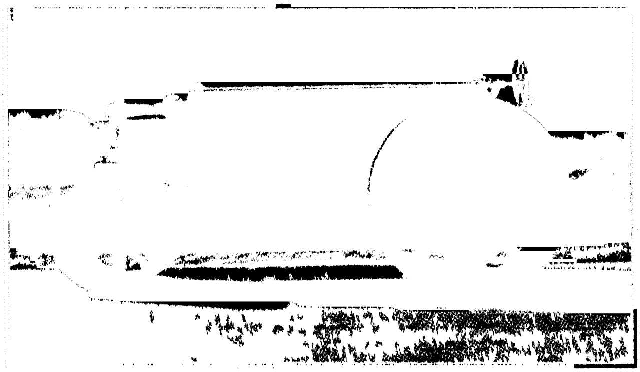


FIG 67 Three-wheel roller, macadam type

A waterbound macadam layer must be allowed to dry out before placing a bituminous surface treatment or wearing course

Maintenance. Inasmuch as a waterbound macadam surface is almost always constructed with a bituminous covering, its maintenance is similar to that of any bituminous surface and will be described in Chapter X

TELFORD FOUNDATIONS

A Telford foundation is constructed by placing flat pieces of ledge stone on the prepared subgrade, filling the interstices with small stone particles, and rolling with a heavy roller. The stone blocks vary from 3 to 8 in. in thickness and from 4 to 15 in. in width and length, and are placed by hand. These foundations have been used on subgrades having low supporting power; they are expensive, and other methods of securing subgrade support are generally more economical.

BITUMINOUS MACADAM PAVEMENTS

A bituminous macadam pavement consists of a lower or base course, usually of dry-rolled broken stone or waterbound macadam, and a wearing course of bituminous macadam. In addition, sub-bases of broken stone, bank gravel, or field stone, are often used on subgrades of low supporting power; Telford foundations are also used. A representative cross-section is shown in Fig. 65.

The stability of the bituminous macadam layer depends upon the quality of the mineral aggregate, the thorough interlocking and compaction of the mineral particles, the cementing ability and quantity of bituminous material, and the thorough sealing or waterproofing of the surface, in addition to good subgrade support. The ideal mineral aggregate for bituminous macadam should be reasonably uniform in size, and hard and tough, so that, after compaction, the surface will be "open" and will allow the bituminous material to penetrate the layer thoroughly. A graded coarse aggregate (that is, one composed of several sizes of particles) has small surface openings and void spaces and does not permit free penetration of the bituminous material; similarly, a soft aggregate tends to fracture under a heavy roller and close up the surface, thereby preventing good penetration of the bituminous material. The bituminous macadam layer is consolidated by means of heavy rollers, and additional interlocking of the stone fragments is obtained, after the layer of coarse aggregate has been placed, compacted, and penetrated with bituminous material, by applying a small quantity of smaller stone fragments (known as *keystone, choke stone,* or *filler stone*), which are wedged into the surface voids by means of rollers. The surface is finally sealed by applying a thin seal coat or cover coat of bituminous material and small stone chips.

Hard stone is not available in many areas, and soft stone is successfully used by varying the procedure for hard stone.

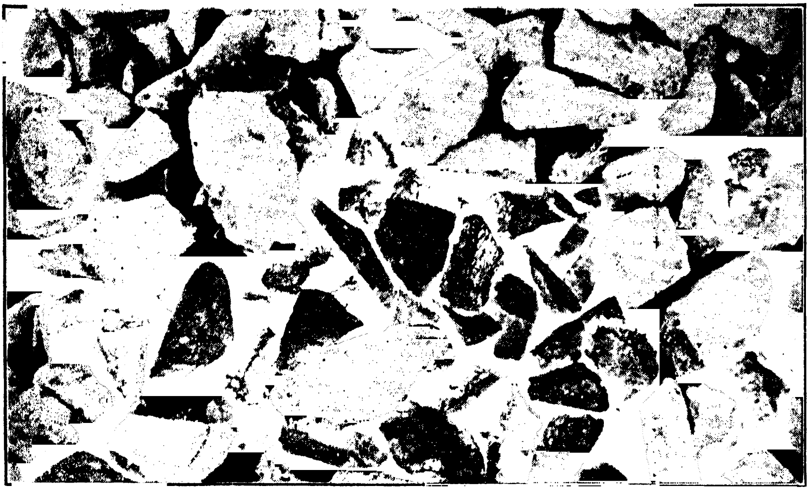
Heavy asphalts and heavy tars, which must be heated for application, are used for the *hot-penetration* construction of bituminous macadam, and emulsified asphalt is used for *cold-penetration* construction.

HOT-PENETRATION CONSTRUCTION WITH HARD STONE

The successive operations in the construction of a bituminous macadam wearing course by the hot-penetration process, after the base course has been constructed, are as follows:

1. **Placing the coarse aggregate on the base or foundation course.**
2. Thorough compaction of the coarse aggregate by means of heavy rollers.
3. Application of the bituminous material by means of pressure distributors.
4. Spreading and rolling of the filler stone.
5. Application of the bituminous material for the seal coat.
6. Spreading and rolling of the stone chips for the seal coat.

Crushed Stone. The term "hard stone," as used in this book, refers to hard varieties of granite and traprock, particularly, and may include some limestones and quartzites. The quality of stone is illustrated by the practice of the Division of Roads and Bridges of the Department of Public Works of Rhode Island in specifying a maximum percentage of wear of 2.7 (in the Deval abrasion machine) and a minimum toughness [2] of 15. Comparatively small stone is used for the coarse aggregate because it is sufficiently strong to resist fracture under a heavy roller and a smoother surface is more easily obtainable with small stone than with large stone; a product ranging in size from 1¼ to 2¼ in. (measured by square opening sieves) is commonly used. The filler stone must be large enough to act effectively in



Courtesy of Rhode Island Board of Public Roads.

FIG. 68. Uniformly graded crushed stone necessary for uniform penetration of bituminous binder.

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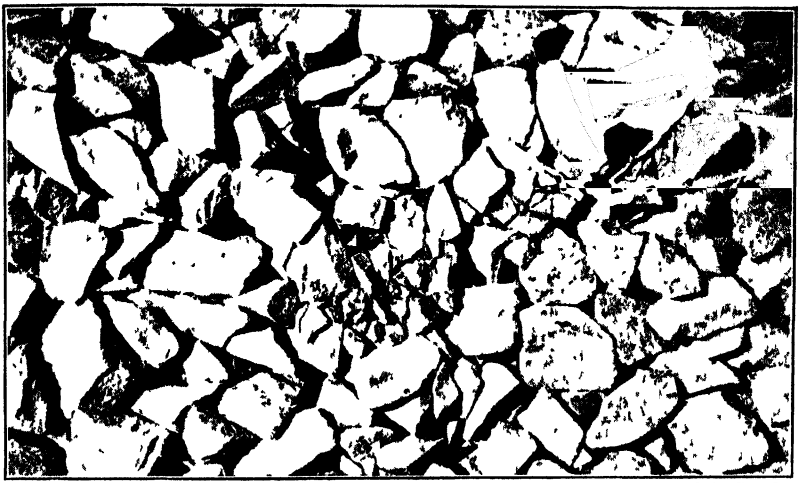
[2] See *Standard Specifications*, American Association of State Highway Officials, for a description of this test.

wedging between the particles of coarse aggregate after the latter have been placed, rolled, and penetrated with bituminous material; a product varying from 1 in. or $\frac{7}{8}$ in. to $\frac{1}{2}$ in. (square openings) is widely used. The stone chips generally vary from $\frac{3}{4}$ or $\frac{1}{2}$ in. to a No. 4 sieve opening in size.

Bituminous Material. The grade of bituminous material for hot-penetration construction should vary according to climatic and traffic conditions. The three grades of oil asphalt most widely used in the United States embrace consistencies ranging in penetration (at 77° F) from 70 to 85, 85 to 100, and 100 to 120. The first grade, 70-85 penetration, is suitable for heavy traffic conditions in the southern states; 85-100 penetration asphalt, for moderate to heavy traffic conditions in the northern states and for light traffic conditions in the southern states; 100 to 120 penetration asphalt, for light traffic conditions in the northern part of the United States. Native lake asphalt is generally restricted to Bermudez Lake asphalt cement (i.e., it is fluxed), and a slightly softer product, as determined by the penetration test at 77° F, is used in comparison with oil asphalt. Corresponding grades of oil and Bermudez Lake asphalt are as follows:



Courtesy of Rhode Island Board of Public Roads.

Fig. 69 Breakage of soft stone under roller. Unless corrected, the bituminous binder will not penetrate uniformly and a "fat" spot will occur.

Oil Asphalt	Bermudez Lake Asphalt
70- 85	85-100
85-100	100-120
100-120	120-150

Three grades of tar are most widely used. Grade RT-11 (see Table 9) is suitable for the northern United States; Grade RT-12 for the southern United States; Grade RT-10 for light to moderate traffic conditions in the northern United States.

Spreading the Coarse Stone. The coarse stone for bituminous macadam requires careful spreading to secure uniform thickness, a smooth surface, and freedom from segregation of sizes. Spreader boxes are desirable, and some hand spreading with rakes is required to obtain a smooth surface.

Rolling the Coarse Stone. The coarse stone is rolled with a heavy three-wheel roller weighing not less than 10 tons, and rolling is continued until the stone is thoroughly compacted and the surface conforms to the desired cross-section and longitudinal grade. This operation is extremely important in obtaining consolidation and surface smoothness, and all irregularities which develop in the stone layer should be corrected by roughening the surface stone, removing and adding stone where necessary, and rerolling. Corrections are conveniently made before the bituminous material is applied but are very difficult to make afterward.

Application of Bituminous Material. Pressure distributors are used to apply the bituminous material. A pressure distributor (see Fig. 70) consists of

1. A tank for holding the bituminous material. Usually the tank is insulated so as better to control the temperature at which the material is applied.
2. A separate power unit for pumping the bituminous material; sometimes compressed air is used for this purpose.
3. Oil burners for heating the material in the tank.
4. A spray bar, which consists of a pipe with openings into which spray nozzles are fitted; this spray bar is attached to the distributor at the rear and usually consists of several sections so that applications of various widths may be made.

Distributors should be equipped with thermometers and devices for measuring the quantity of material being applied. Pressure distributors commonly are constructed in 600-, 800-, and 1000-gal capacities. (A distributor in operation is shown in Fig. 76.)

The quantity of bituminous material varies in accordance with the thickness of the stone layer. Roughly, the total quantity of bituminous material in gallons, the maximum size of the coarse stone in inches, and the thickness of the coarse stone layer in inches, are approximately the same. If 2½- to 1¼-in. stone is used for the macadam layer, approximately 1¾ gal of bituminous material per sq yd will be required for the first, or penetration, application and ¾ gal of material per sq yd for the seal coat.

Asphalt products are generally applied at temperatures of 300° to 375° F and tar products at temperatures of 200° to 275° F. The stone should be thoroughly dry at the time of application of the bituminous material in order that good adherence of the latter to the stone will be secured. The width of the application generally should be a uniform fraction of the total width of the wearing course; widths up to 20 ft may usually be covered in one application and eliminate the danger of applying too much or too little material at the junction between two applications. Uniform application of the bituminous material is essential, and great care should be exercised particularly at the beginning and at the end of the application of distributor loads of material. Deficient applications at connections with previous applications may be prevented by covering the treated surface a short distance back of the end of the previous application with building paper, and starting the new application on the building paper; when the uncovered surface is reached, the distributor is operating at the normal rate. The flow of the distributor should be cut off before the tank is completely emptied to avoid non-uniform application due to non-uniform flow. Irregular areas, such as portions of curved intersections, which are difficult to cover with a distributor, may be covered with a hand spray attached to the distributor tank.

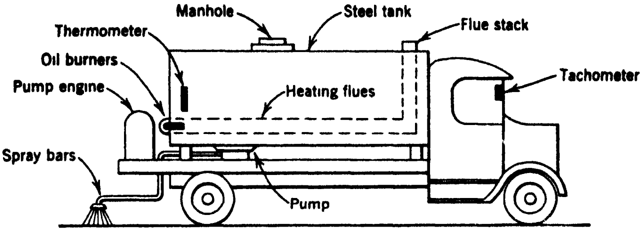
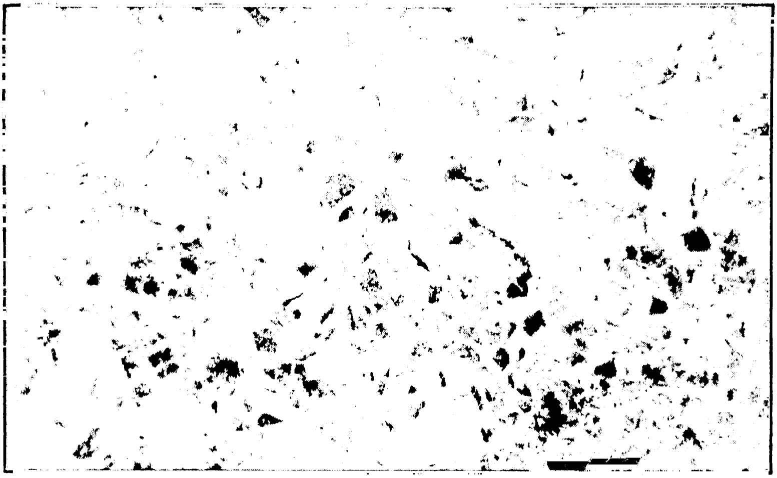


FIG 70 Diagram of pressure distributor.

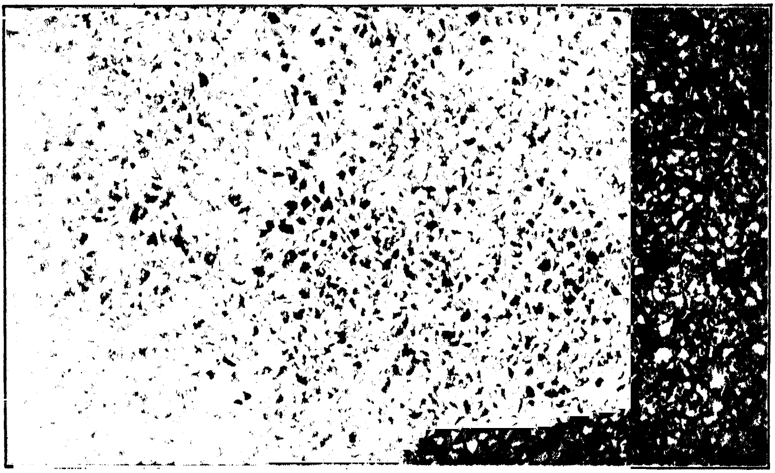
The necessity of correctly determining the proper amount of bituminous material deserves emphasis. An excess of bituminous material destroys the interlocking of the stone fragments and results in a wavy and rutted pavement which can be corrected only by removal and replacement of the wearing course. A deficiency of bituminous material will permit the entrance of water which will tend to destroy the adherence of the stone and bitumen and, if freezing occurs, may destroy the bond between the stone fragments.

Hot-penetration construction is essentially a warm-weather opera-



Courtesy of Rhode Island Board of Public Roads.

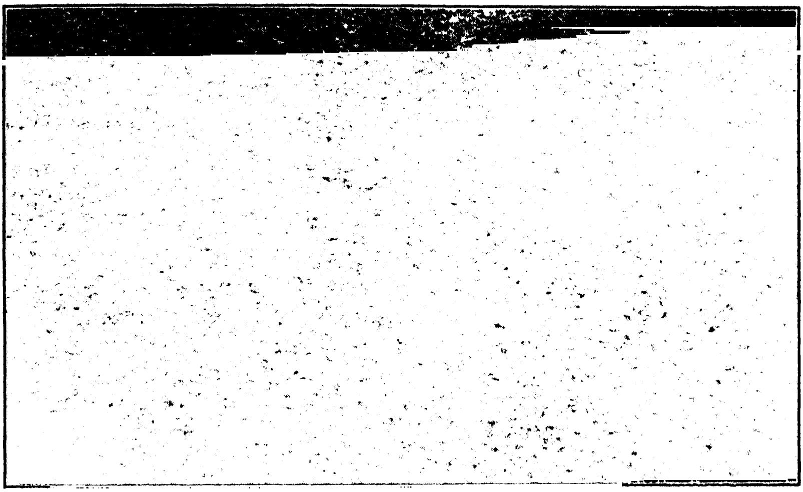
FIG. 71 Proper amount of coarse stone chips or "keystone."



Courtesy of Rhode Island Board of Public Roads.

FIG. 72. Excessive amount of coarse stone chips; they tend to crush under rolling and prevent good adherence of the seal coat.

tion, and the best results will not be obtained if conducted in very cool or cold weather on account of the rapid chilling of the bituminous material as it comes in contact with the cold stone. The North Carolina Highway and Public Works Commission permits the application of bituminous material only when the air temperature is above 50° F. The construction of bituminous macadam is suspended by the Rhode Island Department of Public Works between October 15 and May 10 and by the California Department of Public Works between October 1 and May 1.



Courtesy of Rhode Island Board of Public Roads.

FIG. 73. Surface of seal coat after rolling.

Placing Filler Stone. The stone used for filling the surface voids should be spread immediately after the application of the bituminous material. It is desirable to spread this stone by broadcasting with hand shovels, but mechanical spreaders are sometimes used. The quantity of stone should be just sufficient to fill the surface voids, and 10 to 20 lb per sq yd usually is sufficient; the stone frequently is swept into place with hand push brooms to distribute the fragments most effectively. A surplus of stone should be avoided as it will crush under the roller and form an excessive amount of dust which will prevent adherence of the seal coat to follow. The stone is rolled immediately after spreading so that it will adhere to the bituminous material before it hardens.

Placing Seal Coat. After the filler stone has been spread and rolled, and surface irregularities corrected, a seal coat is applied. An application of bituminous material, at the rate of ½ to 1 gal per sq yd, is made in the same manner as the first application. The stone chips are then spread by mechanical spreaders or by hand broadcasting and the surface is immediately rolled. The quantity of stone chips should be just enough to cover the surface and take up the bituminous material and usually will vary from 15 to 30 lb per sq yd. Sweeping of the surface prior to the application of the bituminous material usually is desirable to remove dust and any excess filler stone that may be present.

HOT-PENETRATION CONSTRUCTION WITH SOFT STONE

The method of hot-penetration construction which has just been described is suitable when comparatively hard stone is available and applies to stone which will not crush seriously while being rolled. As already stated, the practice in Rhode Island utilizes stone having a maximum percentage of wear of 2.7 and a minimum toughness of 15. The Department of Public Works of Massachusetts specifies a maximum percentage of wear of 3 and a minimum toughness of 12. The same construction procedure is followed by the Pennsylvania Department of Highways which specifies a maximum percentage of wear of 5 and a minimum toughness of 6 The Department of Highways of Ohio has developed a successful method of using crushed stone having a maximum percentage of wear of 6 and a minimum toughness of 5; crushed slag having a maximum percentage of wear of 15 and a minimum weight of 70 lb per cu ft is also used. The Ohio method of construction differs from that described for hard stone in the following respects:

1. A slightly larger size of crushed stone, or crushed slag, is used, ranging from 3 in to 1 in , or 2½ to 1½ in., based on square opening sieves, and is placed to a compacted depth of 3 in.
2. Slightly smaller filler stone and cover stone are used, the former being from ¾-in. to a No. 4 sieve opening in size and the latter from ⅜-in to a No 8 sieve opening in size.
3. The coarse stone is not rolled to as great an extent as hard stone.
4. The bituminous material and filler stone are applied to the coarse stone in two applications, rolling following each application of filler stone. The rolling not only embeds the filler stone but also aids in compacting the coarse stone to a greater extent. Cover stone (⅜-in. to a No. 8 sieve in size) may be substituted for the larger filler stone when conditions warrant. The first application of bituminous material varies

from 0.8 to 1.2 gal per sq yd and the second application from 0.6 to 1.0 gal per sq yd with a total of about 1.8 gal per sq yd for the two applications. Each application of filler stone consists of 15 lb of crushed stone or 12 lb of crushed slag per sq yd.

5. A bladed or dragged seal coat is used instead of a plain carpet or cover coat, the crushed stone or crushed slag being applied first, covered with cut-back asphalt, medium tar, or emulsified asphalt, and the two materials being mixed together by means of a long-base blade drag. Thorough rolling follows this mixing process Approximately 30 lb of crushed stone or 24 lb of crushed slag, and 0 6 gal of bituminous material are used for the seal coat.

COLD-PENETRATION CONSTRUCTION

The usual thickness of bituminous macadam wearing courses constructed by the cold-penetration method is 2 in. but may vary from 2 to 4 in. The 1- to 2-in. stone, described for waterbound macadam, is suitable for this method of construction, although other sizes are used. Some engineers prefer a ½- to 1½-in stone and 2- to 3-in. stone is sometimes used when a 3½- to 4-in. layer is to be constructed. Slightly smaller keystone, or coarse chips, and chips are used, the former usually varying from ¾-in. to a No. 4 sieve opening in size, and the latter from a No. 4 to a No 10 sieve opening.

A suitable quick-breaking emulsified asphalt was described in Table 6, Chapter VI.

After the coarse stone is spread and compacted, the voids are filled to within ½ in of the surface with the small chips which are spread by hand broadcasting or by means of a mechanical spreader, rolled, broomed into the voids, and broomed again. A light application of emulsified asphalt is then made and allowed to break down and harden for not less than 12 hours. The surface is then rolled once, the remaining voids are filled with the coarse chips by brooming the stone, and the surface is again rolled. Another application of emulsified asphalt is made, allowed to stand for 12 or more hours, and the surface then is thoroughly rolled. Small chips are then spread over the surface at the rate of 20 to 30 lb per sq yd and broomed and rolled thoroughly. A third application of emulsified asphalt is then made, again allowed to stand for not less than 12 hours, and the surface is again rolled once. Fine sand is next applied at the rate of 8 to 10 lb per sq yd, is thoroughly broomed with a broom-drag (see Fig. 75), and any excess removed by hand or power brooms. The final application of emulsified asphalt is made at the rate of ¼ to ⅜ gal per sq yd and allowed to stand until the "tackiness" has disappeared, when the pavement is opened to traffic. If traffic is allowed to use the pavement while the surface is sticky, a small quantity of sand may be applied to prevent picking up the asphalt.

The quantity of emulsified asphalt will vary with the thickness of the pavement as shown in Table 12.

TABLE 12

QUANTITIES OF EMULSIFIED ASPHALT FOR BITUMINOUS MACADAM WEARING COURSES

Thickness of Pavement, inches	2	3	4
	Gallons per square yard		
First application	½ to ¾	¾ to 1	⅞ to 1⅛
Second application	¾ to 1	¾ to 1⅛	⅞ to 1⅛
Third application	⅓ to ⅜	⅜ to ⅝	⅜ to ⅝
Fourth application (seal coat)	¼ to ⅜	¼ to ⅜	¼ to ⅜
Total quantity	1¾ to 2	2¼ to 2⅝	2⅝ to 3

MAINTENANCE OF BITUMINOUS MACADAM PAVEMENTS

The maintenance of bituminous macadam pavements consists of the replacement of any defective areas which may develop, the correction of depressions by patching, and the occasional renewal of the seal coat. The procedure is very similar to that for bituminous-treated surfaces, which will be described in Chapter X.

BITUMINOUS MACADAM FOUNDATIONS

Bituminous macadam foundations or base courses are generally 3 to 4 in. in thickness and are usually constructed on gravel or crushed stone sub-bases having a thickness of 4 to 6 in. The method of construction is the same as for bituminous macadam wearing courses, except that a smaller quantity (1 to 1½ gal per sq yd) of bituminous material is used for the penetration application, and the quantity of keystone and chips is just enough to fill the surface interstices in the macadam layer without covering the coarse stone itself.

CEMENT-BOUND MACADAM PAVEMENTS

Cement-bound macadam pavements were at one time covered by patents and were known as "Hassam" pavements. The first pavement of this type in the United States was built in Massachusetts in 1905. Development of this type of pavement was slow in the United States, but it was used quite extensively in Europe and Australia. It was used between 1924 and 1930 in numerous cities for paving street-railway track areas, and, about 1930, new interest in it was awakened by economic conditions. Cement-bound macadam pavements are adaptable to roads and streets which are not subjected to heavy truck traffic. A single course of uniform thickness is commonly used, the thickness varying from 6 to 8 in. A crown of ⅛ to ⅕ in. per ft is generally used.

Materials. The materials for a cement-bound macadam pavement consist of a coarse aggregate, sand, Portland cement, and water.

Coarse Aggregate. The coarse aggregate for cement-bound macadam pavements in the United States usually has consisted of broken stone. Broken slag or crushed gravel are suitable coarse aggregate but have been less common than broken stone. Rolling of the coarse aggregate has been considered a desirable feature of the construction operation but it cannot be practiced with screened (and uncrushed) gravel coarse aggregate. Screened gravel has been used, however, without rolling, and it appears to give satisfactory results.

The maximum size of the coarse aggregate generally ranges from 1½ to 2½ in., and a reasonably uniform size is desirable so that the void spaces will be open and permit the cement grout to penetrate and completely fill the layer of coarse aggregate. The two sizes of aggregate (1 to 2 in. and 2 to 3 in.) used for bituminous macadam pavements are satisfactory for cement-bound macadam. The coarse aggregate must be reasonably free from dust and other particles below the minimum size so that, after placing, the grout will not be prevented from penetrating and filling the entire layer.

Sand for Grout. The sound for the cement grout should be clean and uniformly graded from a No. 8 to a No. 100 sieve in size with not over 15 per cent passing a No. 50 sieve.

Cement. Standard Portland cement is used for cement-bound macadam pavements.

Grout Mixture. The grouting mixture consists of 1 part of cement, 1¾ to 2 parts of sand, and 6½ to 7½ gal of water per sack of cement The quantity of water should be sufficient for the grout to flow readily into the coarse aggregate but not great enough to cause the sand to separate from the cement and water. The measurement of the sand by weighing is desirable, but volume measurement, if carefully done, may be satisfactory.

Construction. The subgrade should be carefully prepared as for any type of pavement. Wooden or steel side forms should be used.

The coarse aggregate is spread most satisfactorily by means of spreader boxes. Crushed coarse aggregate is rolled before grouting. Complete compaction is not desired, and the rolling is intended merely to reduce the voids and thereby avoid excess grout. A 3- to 6-ton tandem roller is generally used for rolling. If gravel is uncrushed, heavy straightedges, operated transversely by hand, are used for compaction.

Joints. Longitudinal joints are desirable to eliminate cracking. They may consist of "dummy" joints made by setting a 1-by-2½-in. wooden strip on edge on the subgrade. Transverse joints are not always used but can be constructed in the same manner as the longitudinal joints. Construction joints are made at the end of a day's run by using a bulkhead, the adjacent broken stone being reshaped when work is resumed.

Grouting. The grout may be prepared in truck mixers or in small mixers on the job, or in concrete paving mixers. The broken stone should be lightly sprinkled before the grout is applied. A distributor which will not feed the material too rapidly is used to apply the grout to the broken-stone surface After application of the grout, the surface is rolled out again, preferably with tandem rollers, until a uniform and smooth surface is obtained Additional grout must ordinarily be used, and uneven spots may be trued up with small-size (⅝- to ½-in.) broken stone.

Finishing and Curing. The cement-bound macadam is finished by floating with long-handled, longitudinal floats similar to those used on concrete pavements Excess grout is then swept from the surface by means of brooms or by working wet burlap belts over the surface. After hardening, the pavement should be covered with burlap, which should be kept wet until the pavement is opened to traffic, which, generally, is at the end of seven days after placing.

Maintenance. The maintenance of cement-bound macadam pavements is not essentially different from that of concrete pavements.

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CHAPTER X

BITUMINOUS SURFACE TREATMENTS AND LOW-COST BITUMINOUS SURFACES

Gravel and waterbound macadam road surfaces were ideal for horse-drawn vehicles as the fine dust worn off by the abrasive action of steel tires and by the pounding of horses' hoofs tended to replace the clay or stone dust which formed the binder or cementing material and which was worn off by traffic, blown off by wind, or washed away by rain. The advent of the motor vehicle and the gradual elimination of horse-drawn vehicles created a serious maintenance problem on surfaces of these types. The abrasive action and suction of rapidly moving rubber tires caused great wear and produced large quantities of dust which were a serious inconvenience to traffic and to adjacent property. The condition was most serious, at first, on waterbound macadam roads. Gravel surfaces, composed of round particles of stone mixed with sand and clay, could be reshaped quite conveniently with graders, drags, and scrapers, and the dust nuisance could be greatly lessened by means of surface applications of calcium chloride. Waterbound macadam surfaces, being composed of angular particles of stone, could not be maintained by blading the surface, and bituminous surface treatments were developed, very largely, to prevent wear on surfaces of these types. These treatments consisted of applications of bituminous material which usually were covered with a layer of mineral aggregate. As the speed of motor vehicles increased, the necessity of surface treatment extended to gravel, sand-clay, and traffic-bound macadam surfaces. Early treatments of these surfaces were not always successful because it was difficult to secure consolidation of the upper portions of the surfacing layer and adhesion of the bituminous material to round particles of gravel and fine particles of sand, clay, and crushed stone. The development of bituminous materials for "prime coats" on these surfaces made the use of bituminous surface treatments practicable and effective.

The term *bituminous surface treatment* generally refers to thin surface coverings of bituminous material and mineral aggregate, which are usually less than 1 in. or ¾ in. in thickness. The term also is sometimes used to describe relatively thin *carpet coats* which consist of an application of bituminous material over which is spread a cover of mineral aggregate; this definition distinguishes carpet coats from *road-mix surfaces* which consist of bituminous material and mineral aggregate mixed together on the road surface. Carpet coats are constructed, in some areas, to thicknesses greater than 1 in., and, therefore, the terms *carpet coat* and *road-mix surface* somewhat more accurately describe the process of construction. The term *seal coat* sometimes is used to describe the carpet coat type of surface treatment; generally, however, it is used to describe the covering applied to certain specific types of bituminous surface courses, such as bituminous macadam and asphaltic concrete. Seal coats frequently are not over 3/8 to 1/2 in. in thickness.

Low-cost bituminous surfaces were developed principally for the purpose of resurfacing low and intermediate types of surfaces which no longer would serve satisfactorily as wearing surfaces but which possessed a large amount of salvage value for foundations for new wearing courses. They embrace both *road-mix* and *plant-mix* surfaces, the former being mixed in place directly on the road surface, and the latter being prepared in various types of mixing plants; they should not be confused with high-type bituminous pavements which will be described in Chapter XIII.

BITUMINOUS SURFACE TREATMENTS

Initial bituminous surface treatments usually involve (1) the preparation or conditioning of the surface to be treated, (2) the application of a prime coat, and (3) the placing of the covering of bituminous material and mineral aggregate. They also may include the application of a tack coat.

A *prime coat* is an application of liquid bituminous material to a base course or surface course for the purposes of binding together any loose material which may be present and of providing adhesion to an overlying wearing course or surface treatment. A *tack coat* is an application of bituminous material to a previously primed surface, to a previously treated surface, to a base course, or to an existing pavement or surface course, to provide adhesion with a new wearing course or surface treatment. These two terms are sometimes used interchangeably.

PREPARATION FOR SURFACE TREATMENT

The extent of the preparation for surface treatment depends upon the kind and condition of the surface to be treated.

Stabilized soil surfaces of the bituminous type are surface-treated as the final step in their construction, and preparation for their treatment usually involves only sweeping immediately before the application of the bituminous material.

Firm gravel surfaces are reshaped by means of blade graders while in a moist condition. Deep depressions and defective areas are corrected by the addition of new material which must be thoroughly compacted and smoothed to the proper grade and cross-section. The surface is swept with a power-operated broom just before the application of a prime coat.

Waterbound macadam surfaces are swept clean of all loose dust and dirt before a prime coat is applied. Occasional patching of raveled areas may be necessary if traffic has been permitted upon the surface; it should be performed with the same materials as were used in the construction of the surface

Loosely bound surfaces, such as gravel and traffic-bound macadam, which have been maintained with a floating cover, should be bladed until smooth and until the floating cover is distributed as uniformly as possible. The surface is lightly swept so as not to disturb the firm material below the surface cover, and a prime coat is then applied.

PRIME COATS AND TACK COATS

The bituminous material used for a prime coat must be capable of being absorbed by the surface on which it is placed so that the heavier portion of the material will penetrate well into the surface and not be filtered out and remain merely as a mat on top. Grade MC-1[1] and Grade MC-2 liquid asphalts and Grade RT-2 tars are used as priming materials for loosely bound surfaces; Grade MC-0 liquid asphalt and Grade RT-1 tar are generally used for priming firm surfaces.

The prime coats on firm surfaces, such as gravel and waterbound macadam, usually vary from 0.25 to 0.35 gal per sq yd Prime coats should be completely absorbed, and half of a road or street surface is primed and treated at a time when closing to traffic would be a serious inconvenience. Absorption often takes place in 3 to 6 hours in warm weather, and the surface treatment frequently can be applied on the same day on which the prime coat is applied, although it usually is desirable to allow the prime coat to absorb over night.

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[1] The grade designations of bituminous materials used in this chapter refer to the products described in Chapter VI.

The priming of loosely bound gravel and traffic-bound macadam requires about 0.5 gal of bituminous material per sq yd. The bituminous material should be allowed to absorb completely before traffic is allowed upon the primed surface. If traffic must be allowed to use the primed surface, a cover, which may consist of the fine material swept to the side, should be applied, and dragged with a broom drag (see Fig. 75) until uniformly distributed. Traffic should be permitted to use the primed surface for a period of several days in order to develop any weak places which may exist. These weak areas should be repaired at once by applying additional primer and covering material. When a uniformly firm and even surface is obtained, the surface treatment may be applied. Prime coats are applied with pressure distributors of the types described in Chapter IX.

Tack coats are often put on before the application of bituminous material on surfaces which have been previously treated. Cut-back asphalt of Grades RC-2 and RC-3, emulsified asphalt of the quick-breaking type used for bituminous surface treatment, and tars of Grades RT-3 and RT-4 are satisfactory for tack coats. The quantity of material necessary for a tack coat is very small, usually varying from 0.10 to 0.15 gal per sq yd. Tack coats should be allowed to dry for a few hours before applying the bituminous material for the surface treatment proper; otherwise, the soft bituminous material may be picked up by truck tires.

CARPET COATS

The principal functions of carpet coats are the sealing or waterproofing of the surface on which they are applied and the provision of a wear-resistant surface. They do not aid, greatly, in the removal of surface irregularities and therefore should preferably be used only after the surface on which they are to be applied has been carefully trued up and smoothed by blading or by means of patching or leveling courses.

Bituminous Materials. The essential characteristics of a bituminous material for a carpet coat are: (1) good adhesion to the underlying surface, otherwise the treatment will peel off under the action of traffic and weather; (2) rapid setting or hardening; and (3) stability under traffic. Adhesion to the underlying surface may necessitate a prime coat or tack coat, or both. Rapid setting of the bituminous material is obtained by using a heavy residual asphalt or heavy tar which is heated for application and sets up by chilling to air temperature, by the use of a rapid-curing liquid asphalt or tar which sets up as the result of the evaporation of light ingredients, or by using a quick-breaking emulsified asphalt which sets up as the result of the separation of the asphalt and water and the evaporation of the water. Slow- and medium-curing liquid asphalts were formerly used quite successfully when traffic was slow, but they are entirely unsatisfactory now because they do not hold the mineral aggregate cover in place. Traffic "whips" off the aggregate, and the oils spatter over vehicles; even when set up, the treated surfaces tend to bleed and shove in warm weather.

The liquid materials have been popular for carpet coats because they may be applied cold or with moderate heating. Grade RC-1 liquid asphalt may be applied cold in warm weather and is suitable for thin treatments but sets up too slowly for heavy treatments. Grade RC-2 and heavier grades of liquid asphalt are suited to heavier treatments and must be heated for application. Cold surface treatments with tar are made with material of Grades RT-5 and RT-6. Tars of Grades RT-7, RT-8, and RT-9 are generally used for hot surface treatments. The soft asphalt cements (that is, residual asphalt having a normal penetration of more than 100) are, perhaps, not as widely used as at one time but are very satisfactory materials. Various grades of soft asphalt cements are used (see Tables 7 and 8, Chapter VI), the softer grades for light traffic and the heavier grades for heavy traffic. Asphalt of 85 to 100 penetration occasionally is used for heavy-traffic roads and streets.

Mineral Aggregates. Mineral aggregates for surface treatment include crushed stone, crushed slag, crushed gravel, and uncrushed (but screened) gravel. Crushed aggregates are generally more suitable than rounded aggregates because they resist displacement more effectively. On the other hand, uncrushed gravel is somewhat more suitable than crushed aggregates for thin seal coats which are intended primarily for filling the voids in open-textured surfaces composed of crushed aggregates. The quality of crushed stone aggregates is usually controlled by means of the Deval abrasion test, limiting percentages of wear being widely taken as 6 to 7. Crushed slag should weigh not less than 75 lb per cu ft. The quality of gravel is more difficult to specify but usually may be satisfactorily controlled by restricting the amount of soft particles, clay, lumps, shale, and coal to 5 per cent, or less, by weight.

The sizes of mineral aggregate vary widely, including ¼ to ½ in., ¼ to ⅝ in., ¼ to ¾ in., ¼ to 1 in., and ¼ to 1¼ in. A smaller size of ⅛ to ⅜ in. is occasionally used for very thin treatments. Uncrushed gravel particles over ½ in. in size lack stability and should be avoided. The size of the aggregate selected will depend upon the consistency or viscosity of the bituminous material; the larger sizes of aggregates go with the heavier and more viscous bituminous materials. Most surface treatments are made with aggregates having a maximum size of not more than ¾ in. If sizes are larger, it is desirable to use two sizes (such as ¼ to ¾ in., and ¾ to 1¼ in.) and place them in two applications, the larger first and the smaller last, part of the bituminous material being applied between the two applications of aggregate; a more uniform distribution of sizes is obtained and better adhesion is secured.

The rate of application of the mineral aggregate cover usually is described in terms of the number of pounds of material per square yard of surface. A rather definite relationship exists between the quantity and the maximum size of the aggregate. For example, if a cover 1 in. thick is desired, the maximum size of aggregate should not greatly exceed 1 in. Conversely, if aggregate having a maximum size of 1 in. is to be used, and a rather dense surface is desired, the quantity of aggregate should be such that the thickness of the layer will be approximately 1 in. In other words, the maximum size of the aggregate should coincide approximately with the thickness of the cover. If this relationship between the maximum size and the quantity of aggregate is observed, the quantities of aggregate for various sizes of aggregates will be approximately as shown in Table 13. A variation from this practice occurs when a non-skid surface is desired on a dense, slippery road or street surface. A light application of bituminous material, usually not more than 0.25 gal per sq yd, is applied and ¾ in. crushed aggregate added at the rate of approximately 25 to 35 lb per sq yd.

Construction of the Carpet Coat. The amount of bituminous material required for a plain carpet coat or seal coat depends upon the quantity of mineral aggregate to be used, upon the condition of the surface being treated, and, to some extent, upon the kind of bituminous material. As a general rule, 0.10 gal of bituminous material is required for each 10 lb of aggregate. Slightly greater quantities of the lighter liquid materials may be desirable, especially when the surface being treated is open in texture. Larger proportions of emulsified asphalt generally are required and should be based on the asphalt content of the emulsion.

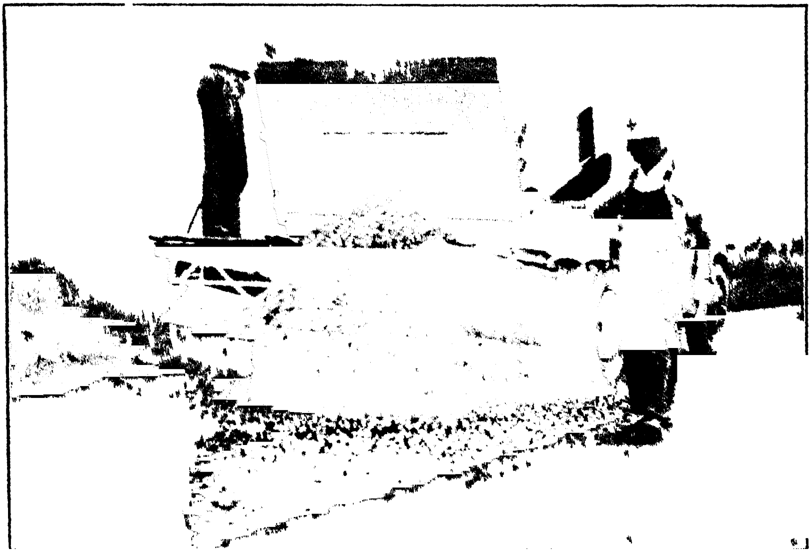
The bituminous material is applied by means of pressure distributors of the types described in Chapter IX.

TABLE 13

Approximate Quantity of Mineral Aggregate for Bituminous Carpet Coat

Types of Aggregates		Crushed Limestone	Round Gravel	Crushed Slag
Weight per cubic yard, pounds		2500	3000	2100
Max Size, inches	Thickness of Cover, inches	Quantity per Square Yard, pounds		
1¼	1	70	83	58
1	⅞	61	73	51
¾	⅝	44	52	32
⅝	½	35	41	29
½	⅜	26	31	22

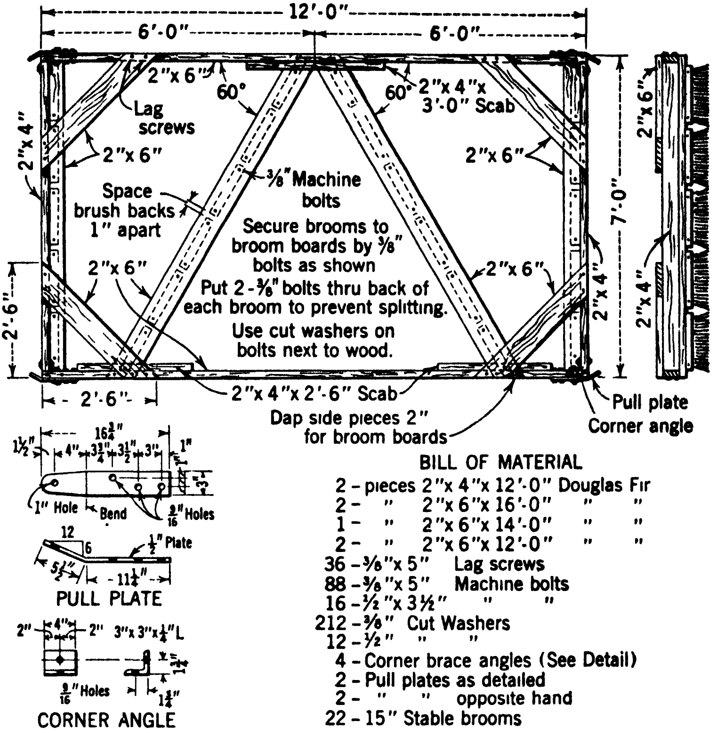
The bituminous material should be covered with the mineral aggregate immediately after application to secure good adhesion The



Courtesy of Universal Crane Co

Fig 74 Placing mineral cover on surface treatment by means of a mechanical spreader.

mineral aggregate may be spread directly from trucks, and good results may be obtained by well-trained truck drivers when truck bodies of uniform width are available. Narrow strips can be spread directly from dump bodies by inserting boards of suitable length in front of the tail gates. There are several types of mechanical spreaders. One consists of a vane or roller over which the aggregate flows in a thin sheet of the desired thickness; another is equipped with a hopper which feeds the aggregate onto a whirling steel plate which throws the material over the road or street surface. Trucks should be moved backwards when applying the mineral aggregate in order to prevent the rubber tires from picking up the freshly applied bituminous material.



BILL OF MATERIAL

2 - pieces 2"x 4"x 12'-0" Douglas Fir
2 - " 2"x 6"x 16'-0" " "
1 - " 2"x 6"x 14'-0" " "
2 - " 2"x 6"x 12'-0" " "
36 - ⅜"x 5" Lag screws
88 - ⅜"x 5" Machine bolts
16 - ½"x 3½" " "
212 - ⅜" Cut Washers
12 - ½" " "
4 - Corner brace angles (See Detail)
2 - Pull plates as detailed
2 - " " opposite hand
22 - 15" Stable brooms

Fig. 75. Broom-drag.

Rolling of the aggregate cover is desirable, although it sometimes is omitted. A light roller weighing from 4 to 6 tons is satisfactory as the only purpose of rolling is to embed the aggregate in the bituminous material so that it will not whip off under traffic. To distribute the mineral aggregate uniformly over the surface a broom-drag (see Fig 75) is frequently used after the first rolling. The broom-drag may be hauled by a truck or rubber-tired tractor. A second rolling should follow the dragging.

Time is a very important factor in the construction of surface treatments. Applications of bituminous materials should follow sweeping operations promptly so as to avoid the necessity of additional sweeping. The mineral aggregate should be spread very soon after the application of the bituminous material for the carpet coat. Asphalt cements, heavy tars, and heavy cut-back asphalts congeal rapidly, and better adhesion of the mineral aggregate will be secured if it is spread immediately after the bituminous application. If any traffic is allowed on the road or street, it should be kept off the freshly applied bituminous material, as the vehicle wheels will pick up the bituminous material and prevent the mineral aggregate from adhering to the road or street surface. Some highway departments apply only enough bituminous material to be covered by the aggregate trucks on hand. This practice delays the aggregate trucks if only one distributor is available. Two distributors, or one distributor and one or two separate supply tanks (mounted on trucks), are desirable when the bituminous material must be hauled over distances in excess of a mile.

Cotton Fabric Reinforcement. Cotton fabrics, sometimes used to reinforce bituminous carpet coats, have proved beneficial. The fabrics are loosely woven and vary in width and weight, an 82-in. width being common. The fabric is applied to the road surface after sweeping and, sometimes, after priming. It may be spread longitudinally by hand or from spools mounted on trucks. Each strip overlaps the previously applied strips at the end and side. The carpet coat is then constructed.

BITUMINOUS ROAD-MIX SURFACES

Bituminous road-mix surfaces consist of mixtures of mineral aggregate and bituminous material which are prepared by mixing the materials directly on the road or street by means of blade graders or multiple-blade drags. They differ from carpet coats in that the mineral aggregate is mixed with the bituminous material rather than applied as a cover. Bituminous road-mix surfaces are of two types—the macadam, or open-graded, aggregate type, and the dense-graded aggregate type.

OPEN-GRADED AGGREGATE ROAD-MIX

The open-graded aggregate type of road-mix surface is said to have originated in Pennsylvania and has been frequently described as "retread" because it has been so widely employed for resurfacing old roads. A thin road-mix surface may replace a carpet coat on light- to moderate-traffic roads and is useful for decreasing minor surface irregularities on roads which require retreatment.

The separate steps in the construction of an open-graded aggregate road-mix surface are:

1. Preparation of the road or street surface or base.
2. Application of a prime coat or tack coat to the surface on which the road-mix is to be placed.
3. Spreading the mineral aggregate.
4. Application of the bituminous material for the mixtures.
5. Mixing and spreading.
6. Consolidation.
7. Application of a seal coat.

Foundations. Bituminous road-mix surfaces are widely used to resurface old gravel, traffic-bound macadam, waterbound macadam, and bituminous macadam roads and streets. They also are constructed on new foundations of these types and on stabilized soil foundations.

Foundations should have a minimum thickness of 6 in. and greater thicknesses up to a maximum of 9 or 10 in. where traffic and subgrade conditions indicate their necessity.

Gravel, stabilized soil, and traffic-bound macadam foundations should preferably be surface-treated and placed under traffic for a period of four to twelve months before the construction of road-mix surfaces of this type in order to correct weaknesses which may develop in these types of foundations.

Thickness. Road-mix surfaces of the macadam aggregate type are constructed in thicknesses varying usually from 1 to 3 in., but may be made as little as ¾ in. thick. With thicknesses of more than 2 in., it generally is desirable to place them in two courses in order to assure complete mixing and to obtain a true, even surface.

Surfaces less than 1½ in. in thickness are generally described as bituminous road-mix surface treatments or sometimes as drag or planed surface treatments. Surfaces 1½ in. or more in thickness are generally described as bituminous road-mix surfaces.

Mineral Aggregates. The mineral aggregates for a road-mix wearing surface of this type consist of crushed stone, crushed gravel, or crushed slag. Crushed particles are essential, because much of the stability of the wearing course depends upon the interlocking or frictional resistance of angular particles. Crushed gravel containing not more than 30 per cent of uncrushed particles has proved satisfactory in areas in which crushed aggregates are difficult to obtain.

The maximum size of the mineral aggregate depends upon the thickness of the layer to be constructed; ½- to 1½-in. material is used for thicknesses of 2 in. or more, ¼- to 1-in. material for a thickness of 1½-in., ¼- to ¾-in. material for a thickness of 1 in., and ¼- to ⅝-in. material for a thickness of ¾ in.

The mineral aggregate for the seal coat should vary in size according to the size of the aggregate used in the mixing course. Material varying from ⅛ to ⅜ in. is suitable with 1½- to 1-in. maximum size coarse aggregate. When thin surfaces are constructed with ¾- or ⅝-in. aggregate, the aggregate for the seal coat should vary from ⅒ to ¼ in. in size.

Bituminous Material. Cut-back asphalts, asphalt emulsions, and tars constitute the bituminous material for the mixing application. Grade RC-3 liquid asphalt is, perhaps, most widely used when cut-back asphalt is employed, but Grade RC-2 liquid asphalt also is used, particularly when the work is done in cool weather. An asphalt emulsion of the slow-breaking type is required on account of the mixing operation (see Table 6, Chapter VI). Tars of Grades RT-6 and RT-7 generally are used, but tar of Grade RT-5 may be necessary in cool weather construction.

The mixing course is generally of the same material as the tack coat and the seal coat, although sometimes a slightly heavier material (usually the next heaviest grade) is employed for the seal coat.

Preparation of Foundation. Road-mix surfaces are most effectively constructed on wearing courses or foundation courses which have received a surface treatment sometime before the construction of the road-mix surface. When a new waterbound macadam base is to be surfaced with a bituminous road-mix, the carpet coat sometimes is omitted or a very thin carpet coat used; if the road-mix is to be placed on the prime coat, a somewhat longer period should elapse between the application of the prime coat and the road-mix than for a carpet coat, in order to prevent cutting of the base by the mixing operation.

Old road and street surfaces may require extensive preparation before resurfacing with bituminous road-mixes. The foundation for a road-mix surface should be 1 to 2 ft wider than the surface in order to obtain adequate edge support, and old wearing courses, therefore, frequently must be widened. Additional thickness also is a frequent necessity, particularly when old gravel wearing courses are to be resurfaced. New material should be thoroughly consolidated before the road-mix is applied. Old macadam and other surface-treated wearing courses often have an irregular surface contour and must be reconstructed to a smooth grade and crown in order to obtain the best results from a road-mix surface. Minor irregularities can be removed by means of thin surface patches of heavy asphalt covered with mineral aggregate of the proper size. Depressions having a depth of more than 1 in. can be satisfactorily eliminated by placing a bituminous macadam layer of the required thickness. The size of stone must be regulated according to the depth of the layer; generally, a maximum size of 1½ to 2 in. is desirable, and the leveling material may be placed in two or more courses in deep depressions. Emulsified asphalt is very satisfactory for this purpose but requires the use of a graded stone to prevent the liquid material from running away at the sides; a hand spray is best adapted to the application of the bituminous material (see Fig. 81).

Prime Coats and Tack Coats. Prime coats are essential on new foundations, and a complete surface treatment is desirable before placing of the road-mix, as already stated.

The application of a tack coat to previously treated surfaces, immediately before placing the road-mix surface, is usually desirable, although it sometimes is omitted, particularly if the surface to be covered has not dried out and cracked. A tack coat aids materially in waterproofing the base and in providing adhesion of the road-mix surface to the base. The tack coat is of the same bituminous material as is to be used in the mixing course, and about 0.10 to 0.15 gal per sq yd is generally required.

Placing the Mineral Aggregate. The mineral aggregate may be spread immediately after the application of the tack coat, or, if it is to be applied to a primed surface, it should be spread after the priming coat has been absorbed by the base. Spreading may best be accomplished by means of mechanical spreaders but may be done directly from trucks. When thicknesses are to be greater than 1½ in., the loose aggregate should be harrowed with a disc harrow to distribute the various sizes of particles uniformly and to facilitate drying of the aggregate if it is moist when placed. A blade grader or drag sometimes supplements the discing.

Application of the Bituminous Material. The bituminous material is applied by means of a pressure distributor, and two applications are desirable on thicknesses greater than 1 in. in order to obtain a uniform coating on the aggregate particles and prevent the material from running away at the edges. Mixing is necessary between the two applications. The quantity of bituminous material required for the mixture will vary according to the quantity of aggregate, physical characteristics of the aggregate, and the quantity of volatile material in the bituminous material. A 2-in. compacted layer of broken stone will require about 0.85 to 0.95 gal of bituminous material per sq yd, and, as already stated, the latter material should be put on in two applications, both approximately equal. A thin layer of stone generally will require more bituminous material, in proportion to the quantity of aggregate, than a thick layer. A ¾-in. layer, for example, will require about 0.35 to 0.40 gal per sq yd if a tack coat of 0.10 to 0.15 gal per sq yd has been applied. No exact rule can be laid down for determining the proportion of bituminous material; the aggregate should be thoroughly coated so that it will stick together, and the appearance of the mixture during mixing will indicate changes to be made in the rate of application. The mineral aggregate should not show free surface moisture when the bituminous material is applied.

FIG. 76. Applying bituminous material for a thin bituminous treatment.

Mixing. Mixing of the mineral aggregate and bituminous material is begun immediately after the latter is first applied and is accomplished by means of blade graders or multiple-blade drags. When the bituminous material is put on in two applications, the mixing after the first one should be continued until the bituminous material is well distributed but not too "tacky"; the second application of bituminous material is then made and the mixing continued until a thorough mixture is obtained. The light constituents of the bituminous materials used for road-mix evaporate rather rapidly, and mixing should be completed before the mixture becomes too "tacky."

Blade graders are slow in comparison with multiple-blade drags, which therefore are more desirable for performing the mixing. The ordinary four-wheel, multiple-blade drag, equipped with blades 10 in. in height (see Fig. 77), can be used on thicknesses up to about 1½ in. but is too light for greater thicknesses. A heavier drag, sometimes called a retread machine (see Fig. 78), is most effective for the thicker layers; it usually requires two men for its operation, one to operate the mixing blades and one to operate the leveling blade at the rear of the drag. These drags may be obtained for mixing widths up to 9 and 10 ft, and two or three trips therefore are necessary to mix the material for a two-lane width. The final trip of the drag should leave the surface planed to a uniform cross-section and to a smooth longitudinal profile; this is accomplished by means of the leveling blade at the rear of the drag.

Roads and streets preferably should be closed to traffic during the construction of road-mix surfaces of the open-graded aggregate type. Not only does traffic displace the mixtures, but also the bituminous material sticks to vehicles and is difficult to remove. Half-width construction should be avoided because of the difficulty of obtaining a smooth joint in the center of the wearing surface and should not be attempted unless the shoulders are unusually wide and firm.

Unless proper precautions are taken, the construction of road-mix surfaces of this type may result in the "tracking" of bituminous material on pavements on intersecting roads. This condition, which is most objectionable on concrete and brick pavements, can be avoided by dispensing with the drag operation for a distance of about 40 to 50 ft from intersecting pavements. An additional quantity of material may be mixed with the drag about 40 to 100 ft on each side of the intersection and spread into place adjacent to the intersection with a blade grader or by hand shoveling and raking.

Compaction of the Surface. After the mixture has become quite "tacky," which will occur usually in 2 to 5 hours, depending upon the characteristics of the bituminous material and upon the air temperature, it is compacted by rolling. The tandem-type roller is best suited for this work, but three-wheel rollers also are used. A 6-ton roller

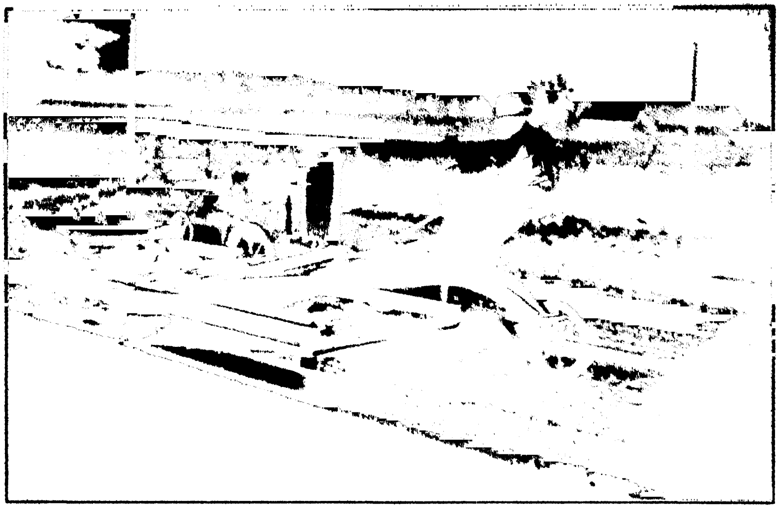
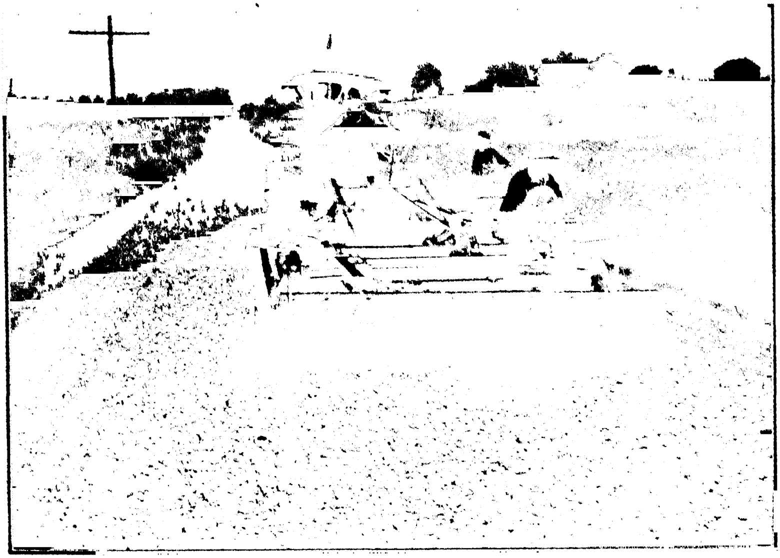


Fig. 77. Mixing a thin layer of bituminous road-mix with a multiple-blade drag



Courtesy of J D Adams Co

Fig. 78. "Retread" machine mixing macadam aggregate road-mix. Rear blade is leveling the surface is satisfactory for thicknesses of less than 1½ in., but 8- to 10-ton rollers are desirable for greater thicknesses.

The edges of thick road-mix courses (1½ in. and more) are likely to be ragged and uneven; this condition may be avoided by having a small gang of men draw in or build up the edges with shovels and tamps immediately ahead of rolling.

Seal Coat. The surface of an open-graded road-mix surface, after rolling, will be quite open and porous, and the placing of a seal coat is necessary in order to prevent raveling under traffic. Complete filling of the surface voids is not practiced, inasmuch as a somewhat non-skid surface is desirable. The quantity of bituminous material and mineral aggregate will vary according to the size of aggregate in the road-mix course. The seal coat for a thin surface course of ¾- to 1-in. thickness generally will consist of 0.15 to 0.20 gal of bituminous material and about 7 to 12 lb of small mineral aggregate per sq yd. The seal coat for 1½- and 2-in. thickness, in which the coarse aggregate is larger, usually will require about 0.25 to 0.35 gal of bituminous material and about 20 to 30 lb of mineral aggregate per sq yd. The seal coat may be constructed in the manner outlined for carpet coats, the mineral aggregate cover being dragged with a broom-drag before rolling. The seal coat also may be road-mixed by means of a blade grader, multiple-blade drag, or truck scraper; a somewhat smoother surface is obtained by this method

The construction of the seal coat usually is deferred from two weeks to several months after the compaction of the road-mix course in order to permit the volatile constituents of the bituminous material to evaporate and leave the surface course fully hardened.

DENSE-GRADED AGGREGATE ROAD-MIX

The dense-graded aggregate type of road-mix was developed in the western part of the United States; it consists of a mixture of bituminous material and an aggregate graded from coarse to very fine particles. Road gravel is the most usual type of aggregate, and mixtures of this kind often have been designated as "oil-gravel road-mix." The gravel in a gravel road surface may be scarified and used for the aggregate or entirely new material may be used.

Foundation and Cross-Sections. Dense-graded aggregate road-mix surfaces are most frequently constructed on old gravel roads or on new gravel foundations. The foundation should generally have a minimum thickness of 6 in., although thicknesses may be and are less on highly stable subgrades. The foundation should be 1 to 2 ft *wider than the road-mix surface, which is constructed to uniform* depths varying from 2½ to 3½ in. A crown of ¼ to 5/16 in. per ft is common.

Mineral Aggregate. The gravel must consist of a mixture of coarse and fine material. Maximum sizes of 1 in. to ¾ in. are desirable, and narrow limitations on grading are not essential as the purpose of this type of construction is to employ materials which are available at low cost. The following limitations on size and grading are representative:

	Per Cent
Passing 1-in. screen	100
Passing ¼-in. screen	50 to 85
Passing No. 10 sieve	35 to 60
Passing No. 40 sieve	20 to 40
Passing No. 200 sieve	5 to 10

Deficiencies in coarse material may be corrected by adding screened gravel or crushed stone. Lack of fine material is corrected by the addition of very fine sand or limestone dust. Some aggregates which have a stronger affinity for water than for bituminous material may be detected by means of the preferential test described in Chapter II.

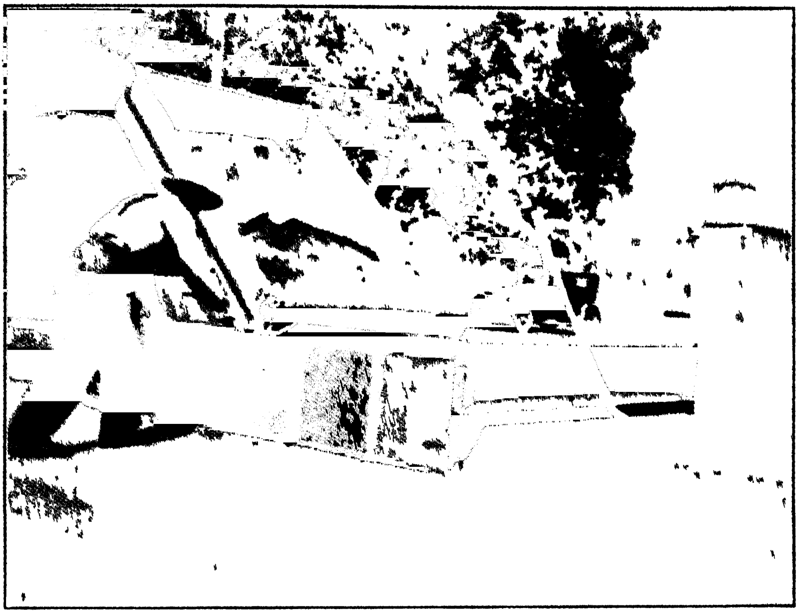
Bituminous Materials. The slow-curing liquid asphalts, Grades SC-2 and SC-3, have been used most extensively for this type of road-mix surface, the former in cool weather and the latter in hot weather. Grades MC-2 and MC-3 liquid asphalt are preferable to the SC products for severe climatic conditions. Grades RT-5 and RT-6 tars are also used. Emulsified asphalt also is widely employed in many parts of the country; a mixing grade, which is slow-breaking, is required.

Preparation of Foundation. The work of preparing the foundation for a dense-graded aggregate road-mix surface may involve widening or increasing the thickness of an existing gravel road, or the construction of an entirely new foundation. When an existing road contains more than sufficient gravel for the foundation, it may be scarified to the desired depth to obtain the quantity of material available from the road itself.

The foundation should be trued up by means of blade graders before the construction of the road-mix course, all scarified or loose material being moved to the sides of the road.

Prime coats sometimes are used with this type of construction and are applied in the manner described for unbonded surfaces earlier in this chapter, unless, of course, the foundation is very firmly bonded, in which case the procedure is as described for bonded surfaces.

Construction of the Road-Mix Surface. After the foundation has been smoothed and trued up by means of blade graders, the loose material is brought in from the sides of the road to form a windrow in the center, and the additional material is then deposited in the center of the road. The correct amount of additional aggregate may be dumped from trucks into spreader boxes which are open in front and have an opening in the rear just sufficient in area to spread the loose material in the proper quantity The windrow of aggregate thus spread is then bladed to a greater width with a grader.

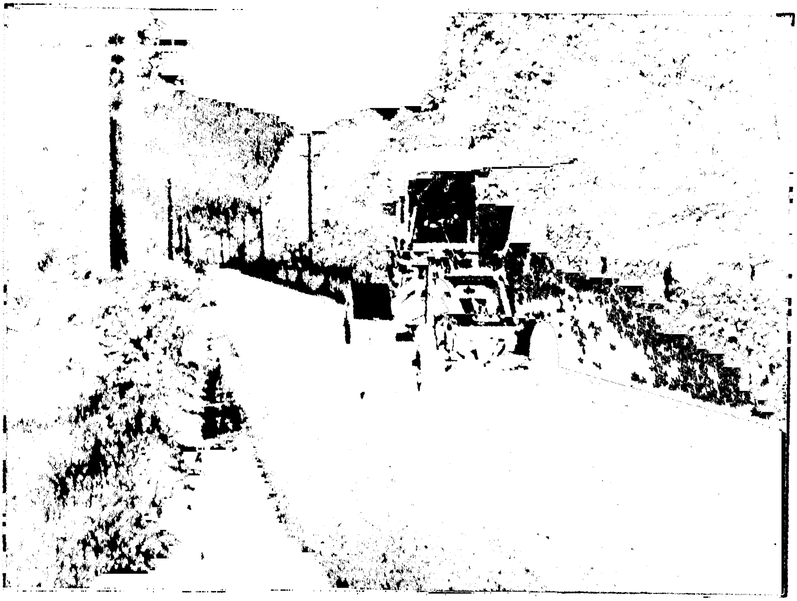


Courtesy of W J Slavin

FIG 79 Proportioning box for placing gravel aggregate on the subgrade for a dense-graded bituminous road-mix, the box is pulled forward by each truck as it deposits its load

It generally is necessary to harrow the gravel with a disc harrow in order to dry it and obtain a uniform mixture When two sizes of aggregate are used, or when a large proportion of the aggregate is obtained from the existing surface, the entire aggregate is mixed with graders and disc harrows until a uniform mixture is obtained. The aggregate should be reasonably dry before the bituminous material is applied; thorough drying is unnecessary when emulsified asphalt is used.

The bituminous material is put on in several applications; two to five are usually necessary, and eight or ten are sometimes made. The use of a number of applications gives an opportunity to adjust the quantity of bituminous material without danger of applying an excess amount. The aggregate and bituminous material are mixed together after each application of bituminous material by means of blade graders and disc harrows, or by means of heavy multiple-blade drags or retread mixers The latter are preferable to graders because they mix the material much more rapidly Liquid asphalt and tar products are usually heated to 140° to 165° F for application Emulsified asphalt is applied without heating



Courtesy of Caterpillar Tractor Co

Fig 80 Mixing graded aggregate road-mix with a motor grader

Road-mix surfaces constructed with slow-curing liquid asphalts are sometimes allowed to compact under traffic, the material, after mixing, being windrowed to the sides and brought in gradually with a grader, as the previously spread material becomes compacted. This procedure is an inconvenience to traffic, and compaction by rolling is desirable, generally by tandem rollers weighing from 8 to 10 tons. Compaction is almost always obtained by rolling when medium-curing liquid asphalt, tar, or emulsified asphalt is used.

The edges of the road-mix surface may be kept straight by having a small crew of men draw them in with shovels and rakes and lightly compact them with hand tamps ahead of rolling. The edge is sometimes thickened by making a V-shaped cut in the foundation with a blade grader before the mixture is finally spread out to full width.

Quantity of Bituminous Materials. Several formulas for determining the proportion of bituminous material in dense-graded road-mixes have been proposed; the earliest formula was developed in California by C. L. McKesson and W. N. Frickstad for slow-curing liquid asphalts; it was as follows:

$$P = 0.015a + 0.03b + 0.17c \tag{33}$$

in which P = the proportion of oil required, expressed as a percentage of the weight of the aggregate.
a = the percentage of the aggregate above the No. 10 sieve, by weight.
b = the percentage of aggregate between the No. 10 and No. 200 sieves, by weight.
c = the percentage of aggregate passing the No. 200 sieve, by weight.

There are several modifications of the McKesson-Frickstad formula; the following modified formula is quite extensively used with slow-curing and medium-curing liquid asphalts, and tar:

$$P = 0.02a + 0.045b + 0.18c \tag{34}$$

The following modification of the formula is appropriate for determining the quantity of emulsified asphalt for dense-graded mixtures:

$$P = 0.05a + 0.1b + 0.50c \tag{35}$$

The proportioning of the bituminous material by means of the foregoing formulas is affected by natural variations in the mineral aggregate, and adjustments of proportions thus obtained are generally necessary during the progress of mixing. Experience with various mineral aggregates and various bituminous materials and the appearance of the road-mix after mixing will indicate changes to be made in the proportion of bituminous material.

Protection of Road-Mix Surface. Dense-graded road-mix surfaces, particularly when constructed with slow-curing liquid asphalts, are comparatively soft in comparison with other bituminous surfaces, and are easily marked by steel tires, horses' shoes, and any object dragged over the surface. These surfaces harden slowly, and the critical period occurs during the first six to ten weeks after placing. Some highway departments, by a program of education and, sometimes, by legal procedure, have obtained the necessary cooperation from road users and adjacent property owners to prevent excessive damage to the surfaces.

Seal Coats. Many dense-graded road-mix surfaces are constructed without seal coats; nevertheless, seal coats are an advantage both from the viewpoint of wear and ease of maintenance. Generally they are thin; 0.25 gal of bituminous material and approximately 15 lb of $\frac{1}{10}$- to $\frac{1}{4}$-in. gravel or stone chips provides a satisfactory seal coat. Grades RC-2, RC-3, and RC-4 liquid asphalt, RT-7, RT-8, and RT-9 tars, and quick-breaking emulsified asphalt are most frequently used. A light application of Grade MC-2 or Grade MC-3 liquid asphalt, covered with sand, also is a very satisfactory seal coat. The seal coat sometimes is bladed with blade graders, truck scrapers, or multiple-blade drags, although it also is applied as a carpet coat.

Maintenance. The maintenance of a dense-graded aggregate road-mix surface involves many of the operations required in the maintenance of most bituminous surfaces, such as replacement of the surface and strengthening of the foundation, surface patching, and the periodical retreatment of the surface. These operations will be described later. The maintenance of these surfaces, however, usually differs in one very important respect from that of other bituminous surfaces. These surfaces tend to dry out, crack, and disintegrate after a period which may vary from five to ten years. They also may become distorted and scaling may occur. When these conditions develop, the most satisfactory procedure appears to consist of completely scarifying the old surface, pulverizing it with disc harrows, adding bituminous material (having the same characteristics as the material in the original construction), mixing, and compacting. This actually is a complete reconstruction, but it can be performed, usually, at very low cost. One-fourth to one-half gallon of bituminous material per square yard will generally be sufficient for the remixing, and the operations involved in reconstruction can be performed rapidly and at a comparatively low cost per mile. The application of seal coats soon after the original construction seems to facilitate this method of reconstruction, probably because the road-mix dries out to a lesser extent than when the seal coat is omitted.

MAINTENANCE OF BITUMINOUS SURFACES

The maintenance of wearing courses having bituminous surface treatments involves the replacement of foundation material, replacement of the wearing surface, surface patching, and periodic surface treatment. The reconstruction of dense-graded aggregate road-mix surfaces as a maintenance procedure at comparatively short intervals of time is peculiar to such types of construction.

The replacement of defective areas of the foundation and of the overlying wearing surface should conform as closely as possible to the original design and construction. The replacement of dense-graded road-mix wearing courses on small areas is facilitated by picking up some of the original mixture at the time of construction and storing it in pits or piles on the roadside; frequently it is possible to use mixtures so stored for periods as long as two to five years.

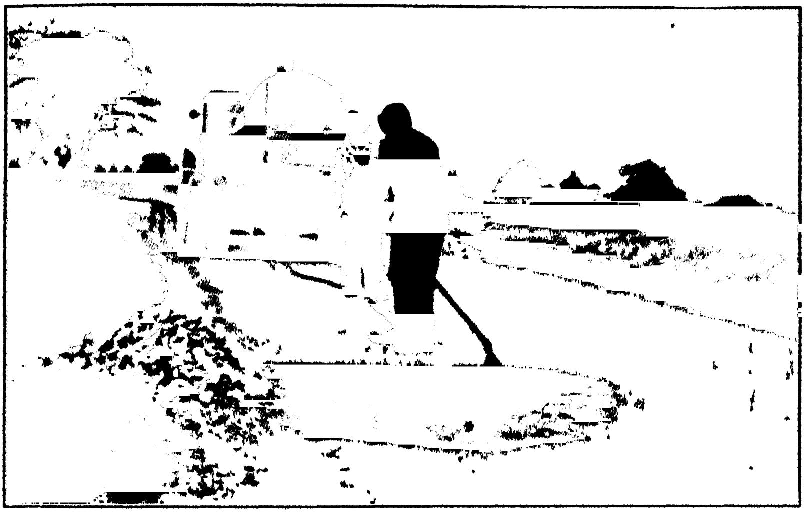


FIG. 81. Applying bituminous material for patching by means of a hand spray.

Bituminous macadam patches, and the replacement of carpet coats and seal coats, may be made with hand sprays attached to kettles or supply tanks. Asphalt cements, cut-back asphalt, tars, and emulsified asphalt are used for patching. The last is convenient because it does not require heating.

Shallow depressions are patched by sweeping them clean and applying a thin tack coat of suitable bituminous material followed by the bituminous material for the cover coat. This is covered with mineral aggregate which is tamped or rolled. Deep depressions may be corrected by means of bituminous macadam patches or by means of plant-mixed material. Heating kettles for making hot patches may be obtained with hoists for handling bituminous material in barrels or steel drums and with pumps and spray bar for spraying the material. Oil or coal serves as fuel, oil being more expensive but affording better control of temperature. Special tanks for heating cut-back asphalts are obtainable, the material being heated by means of oil. Emulsified asphalt possesses the advantage of not requiring heating and is easily applied by power-operated hand sprays.

Plant-mixed bituminous mixtures may be obtained from many commercial plants. A patching mixture, which may be applied cold, may also be prepared in a concrete mixer or in a special type of mixer known as a cold-patch mixer. Such mixtures usually consist of crushed stone and bituminous material; sand is sometimes added but rarely in excess of 10 to 15 per cent of the mixture by volume. A graded crushed stone, ranging in size from ¼ to ¾ in. or from ¼ to 1¼ in., is mixed with 12 to 16 gal of bituminous material per cu yd of stone. Grade RC-2 liquid asphalt and Grade RTCB-5 and Grade RTCB-6 tars are used, and the mixtures should be stored in piles for several days before being applied. They may be kept in storage for several months without deterioration.

Retreatment of bituminous surface-treated surfaces is performed by the methods previously described except that prime coats are not used. Tack coats may be desirable on old and dry surfaces, particularly when heavy bituminous materials are used for retreatment.

LOW-COST BITUMINOUS PLANT-MIX SURFACES

The open-graded and dense-graded aggregate types of road-mix surfaces described in this chapter were developed as low-cost surfaces, and the methods of preparing the mixtures are somewhat inaccurate and subject to numerous limitations Therefore, preparation of the mixtures in mixing plants was a natural development in many locations. Mixing plants for preparing these mixtures include central mixing plants, which are generally installed in stone quarries or gravel pits from which most of the aggregate will be obtained, and traveling mixers, which pick up the aggregate from the roadbed, mix it, and deposit the mixture back on the road surface.

The aggregates in plant-mixed surfaces consist of the same open-graded and dense-graded materials as are used in the construction of road-mix surfaces; crushed oyster shells also have been used to a limited extent. The types of bituminous materials are the same as for road-mix surfaces, but somewhat heavier or more viscous grades of each type are better adapted to plant-mix surfaces.

Preparation of the Foundation. The foundation for plant-mixed surfaces is prepared in the same way as for road-mix surfaces except that scarifying is rarely performed in order to obtain material for the mixture. Prime coats are sometimes used.

Mixing and Placing. The bituminous mixtures are prepared in concrete mixers, standard paving plants, and immersion mixers. Standard paving plants are economical for producing such mixtures because only the bituminous material need be heated, the aggregates being subjected to merely enough heat to remove surface moisture. The immersion type of mixer coats the aggregate by conveying it through a bath of the fluid bituminous material and is used only for coating open-graded aggregate. With concrete mixers and standard asphalt mixing plants the materials may be proportioned by weighing, which results in closer control of the mixtures.

Mixed materials frequently are stored in piles and hauled to the road or street when convenient. Delays due to bad weather often are greatly reduced by this procedure.

Mixtures from central mixing plants are hauled to the road and placed in continuous windrows which are spread and compacted by the same methods as for road-mix surfaces.

Traveling mixers are of several types One type picks up the material from the road surface by means of a bucket loader which discharges it into an overhead bin; a conveyor feeds the material into a mixer where it is mixed with the bituminous material. The mixture then is spread on the road behind the machine in a windrow or, by means of a spreader, to the required width and depth. In another type of machine, the material is picked up by a stationary scoop and is fed directly to the mixer by a screw conveyor; a continuous flow of bituminous material is fed into the mixer, and the machine is equipped with a spreader for leveling the mixture.

Seal coats are an essential feature of the construction of open-graded aggregate, plant-mix surfaces and are preferably used on dense-graded aggregate surfaces.

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CHAPTER XI

THEORY OF STRUCTURAL DESIGN OF CONCRETE PAVEMENTS

The theory of the structural design of flexible pavements was discussed in Chapter VII. Rigid pavements of concrete differ from flexible pavements in two important respects: they possess considerable flexural strength (that is, resistance to bending), and they are affected to a greater degree by changes in temperature.

QUALITY OF CONCRETE FOR PAVEMENTS

Concrete pavements are subjected to the following stresses:

1. Abrasive stresses caused by vehicle tires in contact with the surface of the pavement.
2. Direct compressive and shearing stresses caused by wheel loads.
3. Compressive and tensile stresses resulting from the deflection of the slabs under wheel loads.
4. Compressive and tensile stresses caused by the expansion and contraction of the concrete.
5. Compressive and tensile stresses resulting from the warping of the concrete because of changes in temperature.

Experience has shown that the important properties of concrete for pavements are resistance to wear and flexural strength rather than compressive strength or shearing strength. Direct compressive failures are never found in concrete pavements, and shear failures are very rare.

Resistance to Wear. No reliable method of measuring the resistance of concrete to wear which may be correlated with actual results in service has been developed. Experience and numerous investigations have shown that at least a general relation exists between the resistance to wear and the compressive strength of concrete, and the compressive strength therefore serves indirectly as a measure of resistance to wear. It has become quite general practice to require concrete for pavements to have a water-cement ratio of not more than 6 gal of water per sack of cement. Properly proportioned concrete having this water content will have a minimum compressive strength of 4000 lb per sq in. at 28 days.

Flexural Strength. The flexural strength of concrete paving mixtures generally varies from about 650 to 750 lb per sq in. at 28 days.

EXPANSION AND CONTRACTION OF CONCRETE PAVEMENTS

Longitudinal Expansion and Contraction. As a concrete pavement dries out or cools it tends to contract or become shorter. It is not free to move, however, because of the frictional resistance offered by the subgrade, and tensile stresses therefore are set up within the slab.

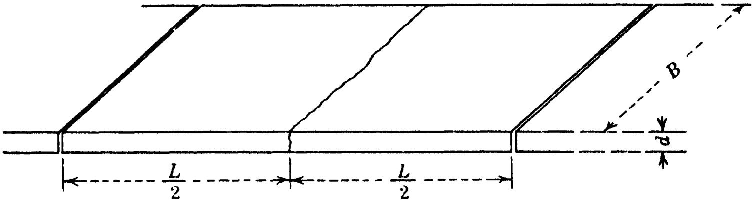


Fig. 82. Spacing of contraction joints in a concrete pavement.

When the tensile strength of the concrete is exceeded a crack is formed. A critical time for crack formation occurs during the period after the concrete has been placed and drying of the slab has begun, because during this period the tensile strength of the concrete is low.

The formation of contraction cracks may be prevented or reduced by constructing contraction joints and by reinforcing the pavement with steel, which has a very high tensile strength. Let Fig. 82 represent a longitudinal section of a concrete pavement of length L and width B. The slab will tend to crack at the middle of its length if the tensile strength of the concrete is uniform throughout its length. The force which produces tensile stresses in the slab is equal to the weight of half the slab times the coefficient of friction between the bottom of the slab and the subgrade. This force must be resisted by the strength of the slab. The Goldbeck formula for equating these forces is as follows:

$$f\frac{L}{2}WB = 12BdS + \frac{E_s}{E_c}aS \qquad (36)$$

in which L = the length of the section, in feet.
B = width of the pavement, in feet.
W = weight of the concrete slab, in pounds per square foot.
d = the average thickness of the pavement, in inches.
S = the allowable tensile stress in the concrete, in pounds per square inch.

E_s = modulus of elasticity of the steel.

E_c = modulus of elasticity of the concrete.

a = area of steel in the pavement cross-section, in square inches.

f = coefficient of friction between the subgrade and the slab.

Since $W = \frac{d}{12} \times 144$ (approximately)

formula 36 may be written

$$6fLdB = 12BdS + \frac{E_s}{E_c} aS \tag{36a}$$

If our purpose is to ascertain the distance between contraction cracks, formula 36*a* may be written

$$L = \frac{12BdS + \frac{E_s}{E_c} aS}{6fdB} \tag{37}$$

If the longitudinal steel is omitted, formula 37 becomes

$$L = \frac{2S}{f} \tag{38}$$

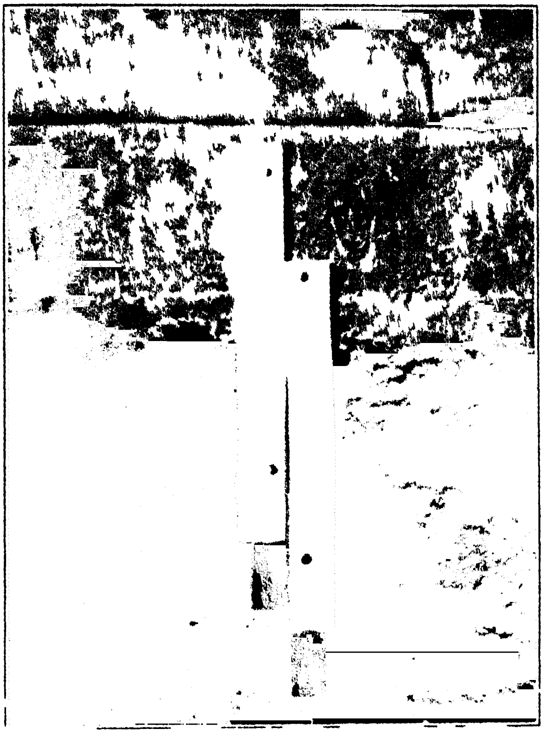
If we consider that narrow cracks are not seriously detrimental to concrete pavements, the spacing of contraction joints will be determined by the maximum allowable unit stress in the steel. Formula 37 will then become

$$L = \frac{aS_s}{6fdB} \tag{39}$$

in which S_s is the allowable tensile stress in the steel, in pounds per square inch. Goldbeck found that the coefficient of friction between concrete pavements and subgrades varied approximately from 0.5 to 2.5 with an average value of about 2.0. The value of S usually assumed is 30 lb per sq in., which provides a factor of safety of 4 or more. The ratio E_s/E_c is approximately 6.0. The value of S_s should be well below the yield point and should vary according to the class of steel; values of 30,000, 25,000, and 20,000 lb per sq in. may be assumed for cold-drawn steel wire, rail steel, and structural steel, respectively. The spacing of contraction cracks in a pavement 18 ft wide and having an average depth of 7 in. would be as follows:

No reinforcement	30.0 ft
Reinforced with No. 3 wires, spaced 6 in. apart	30.2 ft
Reinforced with No 3 wires, spaced 6 in apart, narrow cracks in concrete between contraction joints	33.2 ft
Reinforced with ½-in. round, rail steel reinforcing bars, spaced 18 in. apart, narrow cracks in concrete between contraction joints	38.9 ft

A rise in temperature will cause a concrete pavement to expand, and if the expansion is restrained compressive stresses will be created If the rise in temperature is 60° F, and the concrete has a modulus of elasticity of 5,000,000 lb per sq in and a coefficient of expansion of 0 000 005 per degree Fahrenheit, the direct compressive stress created, if the expansion occurred in a short period of time, would be 1650 lb per sq in This unit stress should not seriously affect the concrete Actually, large changes in temperature, accompanied by great expansive stresses, rarely occur in short periods of time, and the expansive stresses may be reduced by the plastic flow of the concrete This reduction of stress may be offset to some extent by the "growth" of the concrete over a long period of time and by the effect of moisture absorbed by the slab Highway designers commonly provide for expansion joints at spacings which are some multiple of



Courtesy of Division of Highways Illinois Department of Public Works

FIG 83 Illustration of warping of concrete pavements due to changes in the air temperature

the spacing of contraction joints, 90 to 100 ft is the usual spacing, with joint openings of ¾ or 1 in Contraction joints generally are spaced at intervals of 30 to 33 ft apart

Warping of Concrete Pavements. A T Goldbeck of the U S Bureau of Public Roads, and Clifford Older, of the Division of Highways of the Illinois Department of Public Works, discovered independently in 1920 and 1921 the effect of temperature changes in causing concrete pavements to warp or "curl" During the day the top surface of a concrete pavement is heated by the direct rays of the sun, and the temperature in the upper portion of the slab rises much higher and much more rapidly than the temperature in the lower portion of the slab As a result, the upper portion of the slab expands to a greater extent than the lower part of the slab, and the slab warps or curls downward The reverse action takes place at night when air temperatures decrease rapidly The fact that the amount of curling is not negligible but may be sufficient to decrease largely, if not eliminate completely, the subgrade support at the edges of the pavement during the period of contraction is illustrated in Fig 83, which is a reproduction of a photograph taken at the edge of an 18-ft concrete pavement (without longitudinal joints or cracks) at

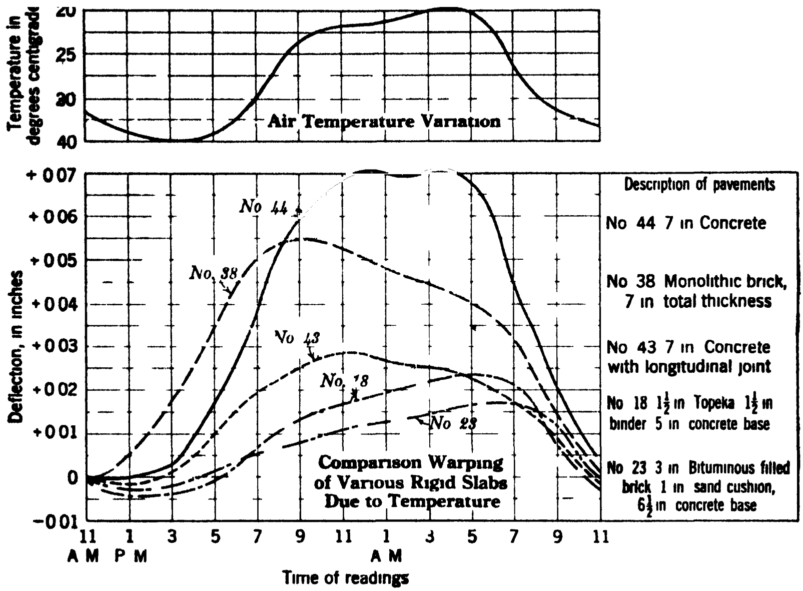


FIG 84 The warping of various types of rigid pavements caused by daily temperature variations (*Clifford Older Trans Am Soc C E, 1924, Vol 87, p 1180*)

night when contraction had taken place. Two paper-covered boards were placed adjoining each other at the edge of the pavement from which the earth shoulder had been removed, one of the boards being securely fastened to the pavement and the other to the subgrade. A continuous line was drawn across each board at the hottest portion of the day. The distance between the lines in the figure was the upward deflection or curling of the edges of the pavement. Heavy loads occur on pavements in the night as well as in the day and induce bending of the slab; when the bending stress exceeds the strength of the concrete slab, fracture occurs and an unsightly crack appears in the slab.

Older found that the extent of transverse warping varied according to the width of the concrete slab, and that it was reduced more than 50 per cent in two-lane pavement slabs by using a longitudinal center joint (see curve 43, Fig. 84).

Since transverse warping of concrete pavement slabs occurs with changes in temperature, it follows that warping would occur in all directions were it not for the restraint offered by the slabs in a lengthwise direction. The stresses developed by this restraint were generally neglected in the design of concrete pavements because it was believed that their magnitude was not great enough to affect materially the structural strength of pavement slabs. Stress measurements made by the U. S. Public Roads Administration under very carefully controlled conditions, and reported in 1935 and 1936, showed that the magnitude of these stresses was sufficient to cause cracking of the slabs when loaded, and indicated that joints to control warping should be spaced at intervals of about 10 ft. The magnitude of temperature warping stresses will be discussed later in this chapter.

THICKNESS OF CONCRETE PAVEMENTS

Older's Stress Analysis. Clifford Older, formerly Highway Engineer of the Division of Highways, Illinois Department of Public Works, was probably the first engineer to design concrete pavements on the basis of their analysis as a structure provided to withstand certain loads. Older concluded from extensive observations that the critical point in a concrete pavement is at the intersection of a transverse joint or crack and the outside edge of the pavement, and that concrete pavements could be designed, as structures, to resist the stresses produced by vehicle loads. Figure 85 illustrates Older's procedure. The load W is applied at the outside corner of a section of slab formed by a transverse joint or crack. The slab will tend to

fracture along a line *AB* at some distance, *a*, from the corner. Observations of corner breaks show that the line *AB* may be assumed to be at an angle of 45° with the edge of the pavement. The bending moment produced by the load W is

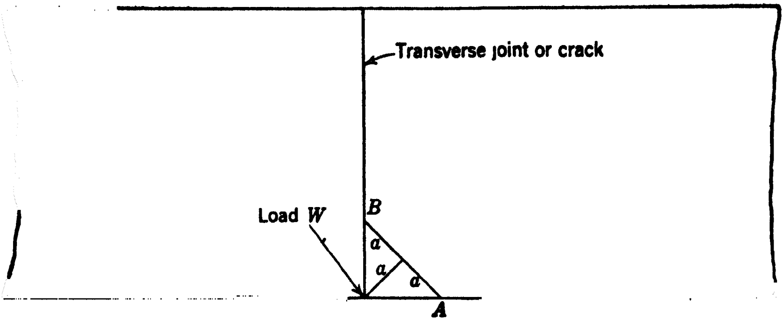


FIG. 85. Critical condition for fracture of a concrete pavement at edge of slab.

$$M = Wa$$

which must be counteracted by the resisting moment of the slab. Therefore,

$$Wa = \frac{SI}{y} \tag{40}$$

in which S = the tensile stress in the outer fiber of the concrete, in pounds per square inch.
I = the moment of inertia of a rectangular vertical section of the slab along the line AB, about the neutral axis of the section, in inches
y = the distance of the outer fiber of the concrete from the neutral axis of the vertical section of the slab along the line AB, in inches.

The moment of inertia of the vertical section of the slab along the line AB is

$$I = \frac{1}{12} \times 2a \times d^3 = \frac{ad^3}{6}$$

in which d = the depth of the slab, in inches. The neutral axis of the vertical section of the slab is approximately at the center, so that y in formula 40 may be assumed to be $d/2$. Therefore, formula 40 may be written

$$Wa = \frac{Sad^2}{3}$$

or

$$d = \sqrt{\frac{3W}{S}} \tag{41}$$

The factor a has disappeared.

It will be observed that this formula refers to a plain concrete slab which receives no support from the adjacent slab. If we now consider the central portion of the slab, we find that the same formula will apply to the critical condition, namely, an interior corner formed by the intersection of longitudinal and transverse joints and cracks. This condition is illustrated in Fig. 86. Corner cracks are likely to occur at any section such as *CD, DE, EF*, or *FC*. Some frictional resistance is set up in many cases between the fractured surfaces which may lessen the stress in one interior corner; but it cannot be depended upon and is too small to be taken into account in the design.

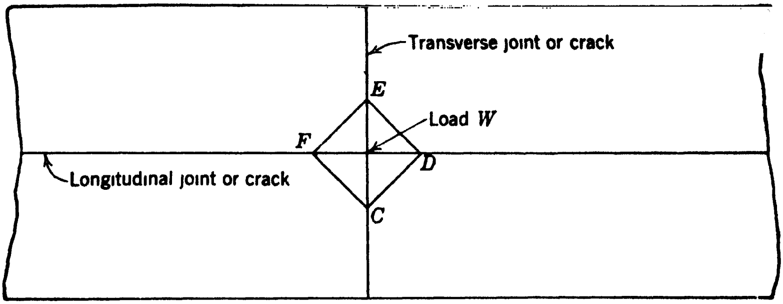


FIG 86 Critical condition for fracture of a concrete pavement in interior of slab, due to load stresses

Older discovered that the deflection of the corners of concrete slabs was materially decreased if the slabs were held together by means of friction between the planes of contact or by means of steel embedded in the concrete (see Fig. 87). Friction cannot be depended upon because both joints and cracks will open up or become wider with decreases in temperature which cause contraction. Figure 87 shows that the deflection was reduced about one-half by embedded steel; therefore, Older assumed that approximately one-half of the load was transmitted across the joint or crack into the adjoining slab, the steel being stressed in shear only. With this condition we may write formula 41 as follows:

$$d = \sqrt{\frac{1.5W}{S}} \tag{42}$$

Returning to an analysis of the critical point in the central portion of the slab, as indicated by the intersection of a transverse joint and a longitudinal joint or of a transverse crack and a longitudinal crack, we may decrease the deflection of each corner by placing steel longitudinally or transversely, or both. If longitudinal steel, only, is placed, and if no transverse support is afforded between adjacent slabs formed by either a joint or a crack, the thickness of the pavement must be determined by formula 42. If, however, transverse as well as longitudinal steel is used, the load will be transferred to some extent in both directions to adjacent slabs The extent to which the load is transferred to the adjacent slabs was open to some question, but Older suggested the use of the formula.

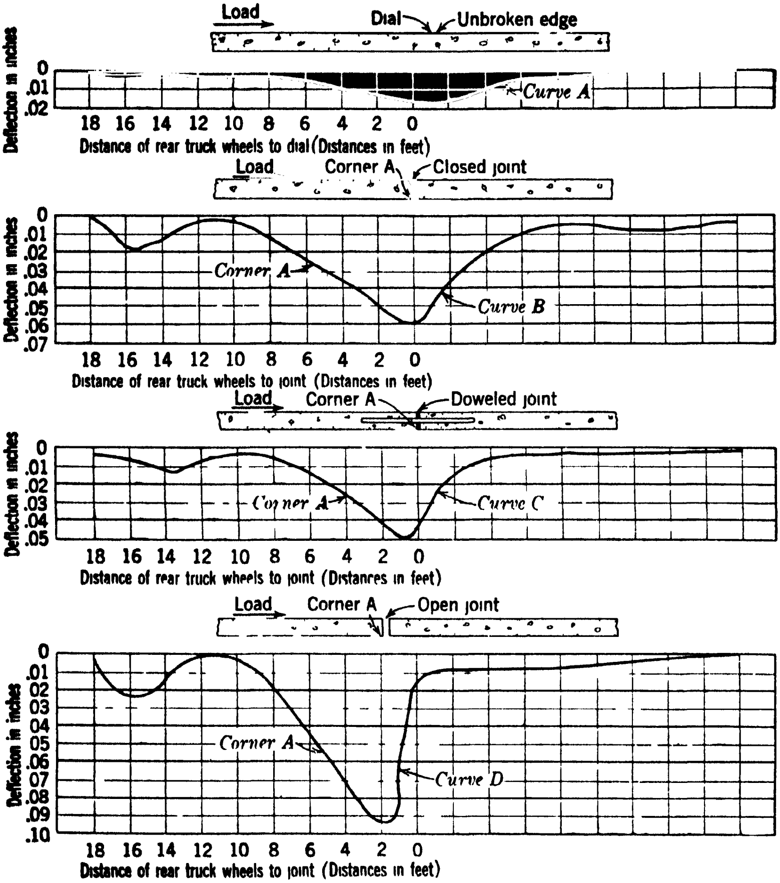
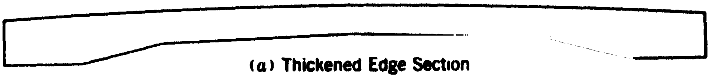


FIG 87 Deflection of a concrete pavement due to a moving load (*Older*)

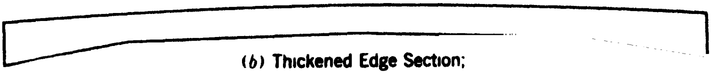
$$d_i = \sqrt{\frac{W}{S}} \qquad (43)$$

in which d_i = the interior depth of the slab, in inches.

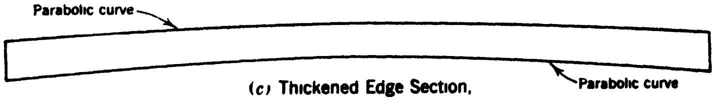
It is apparent, from these considerations, that the thickness of pavements may be reduced if provision is made, by means of steel, to transmit loads across joints and cracks. The width of the thickened edge of pavements containing transverse as well as longitudinal steel remains to be determined. Older observed that corner breaks seldom occurred at distances of more than 2 ft from the corner and concluded that the width of the thickened edge should be 2 ft and that the thickness of the pavement should be reduced to the interior thickness in the next 2 ft, as shown in Fig. 88(*a*). There was some objection to this cross-section on account of the difficulty of shaping the subgrade, and the shape of cross-section shown in Fig. 88(*b*) has been extensively used. Another cross-section provided for the edge thickening by means of two parabolic curves as shown in Fig. 88(*c*). Formulas 42 and 43 may be applied to the latter cross-section without appreciable error, but formula 42 should not be used to determine the edge thickness of pavements having the cross-section shown in Fig. 88(*b*). Practice has varied in regard to the determination of the edge thickness of the trapezoidal edge section of Fig. 88(*b*). A simple solution consists of assuming that the value of *d*, obtained from formula 42, may be taken as the average thickness of the trapezoidal edge section. The outside edge thickness then would be determined from the formula



(*a*) Thickened Edge Section
(Older)



(*b*) Thickened Edge Section;
Wedge Thickening



(*c*) Thickened Edge Section,
Two Parabolic Curves

FIG. 88. Shapes of cross-sections of concrete pavements.

$$d_e = d_i + 2(d - d_i) \tag{44}$$

in which d_e = the thickness, or depth, of the outside edge of the slab.

Since the endurance limit of concrete subjected to repeated applications of loads is approximately 55 per cent of the ultimate strength, and since, as already stated, paving concrete generally has an ultimate flexural strength of 650 to 750 lb per sq in., allowable unit stresses of 300 to 350 lb per sq in. have been widely used in the preceding formulas. Older believed that it was unnecessary to provide for the effect of impact because concrete pavements may be constructed and maintained to eliminate significant impact due to surface irregularities. Many designers, however, have believed that some allowance for impact should be made on account of deflections under loads at joints and cracks, and other conditions which at times would contribute to moderately high impact forces. Older also believed that subgrade support should be disregarded because of the lack of contact of the under side of the pavement slab and the subgrade. The procedure of design will be illustrated by assuming that a pavement will be subjected to a wheel load of 8000 lb (which is the maximum legal load recommended by the American Association of State Highway Officials). Using formulas 42 and 43, with an allowable unit stress of 350 lb per sq in., and providing for an increase of 50 per cent in the load W due to impact, the edge and interior thicknesses of the slab for the cross-section shown in Fig 88(a) would be 7 2 and 5 9 in, respectively. If the edge thickness for the cross-section shown in Fig. 88(b) is determined by means of formula 44, which is approximate only, a value of 8 5 in is obtained. These thicknesses are very close to the 9-6-9 in. cross-section so widely used.

Formula 42 assumes that 50 per cent of a wheel load at the outside edge of the pavement is transferred across a transverse joint when dowels are placed across the joint, and formula 43 assumes that two-thirds of the load is transferred from the loaded slab to the adjacent slabs at interior corners Doweled joints do not provide for this degree of efficiency It probably is safe to assume that one-third of the load is transferred across transverse joints when closely spaced dowels are used across the joint; formula 42 then would be modified to read as follows:

$$d = \sqrt{\frac{2W}{S}} \tag{42a}$$

and, allowing for the same proportion of transfer across longitudinal joints, formula 43 would be modified as follows:

$$d_i = \sqrt{\frac{1.33W}{S}} \tag{43a}$$

Using formulas 42*a* and 43*a*, with an allowable unit stress of 350 lb per sq in., and providing for an increase of 50 per cent in the load W due to impact, the edge and interior thicknesses of a slab carrying a wheel load of 8000 lb, and designed according to the cross-section shown in Fig. 88(*a*), would be 8.33 and 6.76 in., respectively. If formula 44, which, as already stated, is approximate only, is used to determine the edge thickness for the cross-section shown in Fig. 88(*b*), a value of 9.9 in. is obtained. These thicknesses correspond closely to the 10-7-10-in. cross-section which is widely used on heavy-traffic highways.

This analysis of stresses was an outstanding development in the structural design of concrete pavement slabs but was incomplete in the following respects:

1. It was assumed that the critical stresses occurred near the corners of slabs and did not provide a method for determining the stresses in other portions of the slab.
2. The wheel load was assumed to be concentrated at the corner point of the slab, whereas it is distributed over an area of contact between the tire and the slab.
3. Some allowance for subgrade support may be justifiable under some conditions, particularly under the interior portion of slabs.
4. It did not provide a definite procedure for evaluating impact.
5. The magnitude of the stresses developed by restrained warping was not taken into consideration.
6. The establishment of minimum thicknesses for pavement slabs, particularly on roadways subjected to light-weight traffic, was not standardized.

Westergaard Stress Analysis. Professor H. M. Westergaard made theoretical studies of the stresses developed in a concrete pavement by static loads applied at three critical points, assuming that the pavement slab acted as a homogeneous isotropic elastic solid, that the entire slab was in contact with the subgrade, and that the subgrade reactions were vertical and directly proportional to the slab deflections. He expressed the subgrade reaction per unit of area at any given point as a coefficient times the deflection at the point, calling the coefficient the *modulus of subgrade reaction* and denoting it by the symbol k. The resistance of the slab to deflection not only depends upon the subgrade reaction but also upon its own (i.e., the slab's) stiffness, which is a function of its modulus of elasticity, E_c. Westergaard expressed this relation between the stiffness of the subgrade and the slab as a linear dimension, l, which he called the *radius* *of relative stiffness of the slab and subgrade,* and the value of which is determined from the formula

$$l = \sqrt[4]{\frac{E_c d^3}{12(1 - \mu^2)k}} \tag{45}$$

in which l = the radius of relative stiffness of the slab and the subgrade, in inches.

E_c = the modulus of elasticity of the concrete, in pounds per square inch.

d = the thickness of the slab, in inches.

μ = Poisson's ratio of lateral to longitudinal deformation of the concrete.

k = the modulus of subgrade reaction, in pounds per square inch per inch of deflection; assumed constant for a given slab and a given type of loading.

Values of l for various values of E_c, d and k are given in Table 14. The three critical load positions are (1) corner loading, (2) edge loading, and (3) interior loading, and are shown in Fig. 89. In corner loading, the maximum tensile stress occurs in the top of the slab at some distance from the load at the corner, and the maximum bending moment is distributed over a vertical section of the slab at a 45° angle with the longitudinal edge of the slab (such as the section *AB* in Fig. 85). With interior loading and edge loading, different conditions exist. With interior loading, the maximum bending moment and the corresponding maximum tensile stress at the bottom of the slab occurs under the load and raises the question of what "effective" sectional area of the slab resists the tensile stress set up by this maxi-

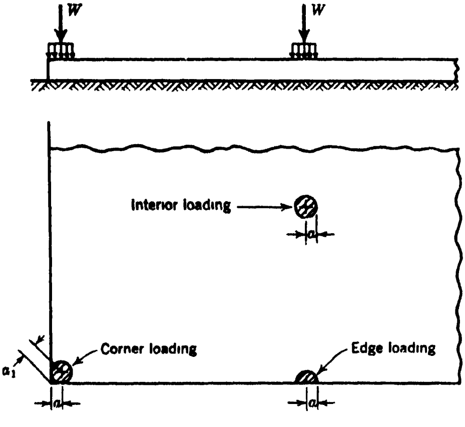


FIG. 89. Westergaard's diagram of wheel loads on concrete pavements

TABLE 14

Values of Radius of Relative Stiffness of Concrete Pavement Slab and Subgrade for Various Values of E_c, d, and k, and for $\mu = 0.15$ in Formula 45

(*U. S. Public Roads Administration*)

Modulus of Elasticity, E_c, lb per sq in.	Subgrade Modulus k, lb per cu in.	Radius of Relative Stiffness, l, in inches, for Different Values of d, in inches.								
		$d = 4$	$d = 5$	$d = 6$	$d = 7$	$d = 8$	$d = 9$	$d = 10$	$d = 11$	$d = 12$
3,000,000	50	23.9	28.3	32.4	36.4	40.2	43.9	47.6	51.1	54.5
	100	20.1	23.8	27.3	30.6	33.8	37.0	40.0	43.0	45.9
	150	18.2	21.5	24.6	27.7	30.6	33.4	36.1	38.8	41.4
	200	16.9	20.0	22.9	25.7	28.4	31.1	33.6	36.1	38.6
	300	15.3	18.1	20.7	23.3	25.7	28.1	30.4	32.6	34.8
	400	14.2	16.8	19.3	21.6	23.9	26.1	28.3	30.4	32.4
4,000,000	50	25.7	30.4	34.8	39.1	43.2	47.2	51.1	54.9	58.6
	100	21.6	25.6	29.3	32.9	36.4	39.7	43.0	46.2	49.3
	150	19.5	23.1	26.5	29.7	32.8	35.9	38.8	41.7	44.5
	200	18.2	21.5	24.6	27.7	30.6	33.4	36.1	38.8	41.4
	300	16.4	19.4	22.3	25.0	27.6	30.2	32.7	35.1	37.4
	400	15.3	18.1	20.7	23.3	25.7	28.1	30.4	32.6	34.8
5,000,000	50	27.2	32.1	36.8	41.4	45.7	49.9	54.0	58.0	62.0
	100	22.9	27.0	31.1	34.8	38.4	42.0	45.4	48.8	52.1
	150	20.7	24.4	28.0	31.4	34.7	37.9	41.1	44.1	47.1
	200	19.2	22.7	26.0	29.2	32.3	35.3	38.2	41.0	43.8
	300	17.4	20.5	23.5	26.4	29.2	31.9	34.5	37.1	39.6
	400	16.2	19.1	21.9	24.6	27.2	29.7	32.1	34.5	36.8
6,000,000	50	28.4	33.6	38.6	43.3	47.8	52.3	56.6	60.7	64.8
	100	23.9	28.3	32.4	36.4	40.2	43.9	47.6	51.1	54.5
	150	21.6	25.6	29.3	32.9	36.4	39.7	43.0	46.2	49.3
	200	20.1	23.8	27.3	30.6	33.8	37.0	40.0	43.0	45.9
	300	18.2	21.5	24.6	27.7	30.6	33.4	36.1	38.8	41.4
	400	16.9	20.0	22.9	25.7	28.4	31.1	33.6	36.1	38.6

TABLE 15

Values of Radius of Equivalent Distribution of Pressure of Concrete Pavement Slabs, according to Formula

$$b = \sqrt{1.6a^2 + d^2} - 0.675d$$

(*U. S. Public Roads Administration*)

Ratio of a/d	Values of b, in inches, for Different Values of d, in inches.								
	$d = 4$	$d = 5$	$d = 6$	$d = 7$	$d = 8$	$d = 9$	$d = 10$	$d = 11$	$d = 12$
0	1.30	1.63	1.95	2.28	2.60	2.93	3.25	3.58	3.90
0.1	1.33	1.66	2.00	2.33	2.66	3.00	3.33	3.66	4.00
0.2	1.43	1.78	2.14	2.50	2.85	3.21	3.57	3.92	4.28
0.3	1.58	1.97	2.37	2.76	3.16	3.55	3.95	4.34	4.73
0.4	1.78	2.23	2.67	3.12	3.57	4.01	4.46	4.90	5.35
0.5	2.03	2.54	3.05	3.56	4.07	4.57	5.08	5.59	6.10
0.6	2.32	2.90	3.48	4.06	4.64	5.22	5.80	6.38	6.96
0.7	2.64	3.30	3.96	4.62	5.29	5.95	6.61	7.27	7.93
0.8	2.99	3.74	4.49	5.23	5.98	6.73	7.48	8.22	8.97
0.9	3.36	4.20	5.04	5.88	6.72	7.56	8.40	9.24	10.08
1.0	3.75	4.69	5.62	6.56	7.50	8.44	9.37	10.31	11.25
1.1	4.15	5.19	6.23	7.27	8.31	9.35	10.38	11.42	12.46
1.2	4.57	5.71	6.86	8.00	9.14	10.28	11.43	12.57	13.71
1.3	5.00	6.25	7.50	8.75	10.00	11.25	12.50	13.75	14.99
1.4	5.43	6.79	8.15	9.51	10.87	12.23	13.59	14.95	16.30
1.5	5.88	7.35	8.82	10.29	11.76	13.23	14.70	16.17	17.64
1.6	6.33	7.91	9.49	11.08	12.66	14.24	15.82	17.41	18.99
1.7	6.79	8.48	10.18	11.88	13.57	15.27	16.97	18.66	20.36
1.724 *	6.90	8.62	10.34	12.07	13.79	15.52	17.24	18.96	20.69

* When a/d is greater than 1.724, b –

mum bending moment. Theoretically, if the load were concentrated at a point, the bending moment under the load would have a value of infinity. Actually, however, the bending moment and the resulting tensile stresses are not infinite in value but are finite values owing to the effect of the slab thickness in distributing the bending moment over a vertical section whose horizontal projection is circular in shape and whose center corresponds with the center of the loaded area.

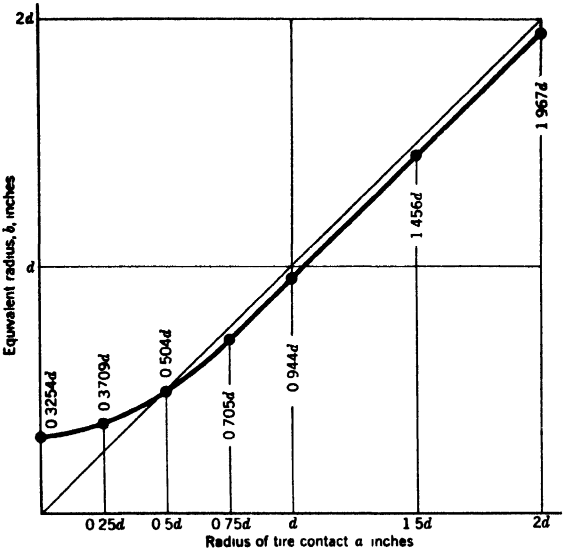


Fig. 90. Relation between the radius of tire-contact area on a pavement slab and the radius of equivalent distribution of pressure of the pavement slab. (*H. M. Westergaard.*)

Westergaard called the radius of this circle the *radius of equivalent distribution of pressure*, and calculated values of it in terms of the thickness of the slab, d, in inches, and the radius of the loaded area, a, in inches. The results of his calculations are shown graphically in Fig. 90. The curve in Fig. 90 closely approximates a hyperbola represented by the equation

$$b = \sqrt{1.6a^2 + d^2} - 0.675d$$

in which b is the radius of equivalent distribution of pressure. For values of a larger than $1.724d$, b may be taken equal to a. Calculated values of b for various values of a and d are given in Table 15. The conditions under edge loading are somewhat similar to those under interior loading, and Westergaard replaced the radius of the loaded area, a, by the radius of equivalent distribution of pressure, b, in formulas developed for calculating maximum tensile stresses induced by interior and edge loadings.

An elaborate program of stress measurements in concrete pavements was undertaken by the U. S. Public Roads Administration at Arlington, Virginia, to determine the correlation between actual stresses and the stresses calculated by means of Westergaard's formulas. As a result of this investigation, E. F. Kelley has recommended the following formulas for determining the maximum tensile stresses in concrete pavement slabs induced by loads:

Corner loading

$$S_c = \frac{3W}{d^2}\left[1 - \left(\frac{a\sqrt{2}}{l}\right)^{1.2}\right] \tag{46}$$

Interior loading

$$S_i = \frac{0.316W}{d^2}\left[4\log_{10}\left(\frac{l}{b}\right) + 1.069\right] \tag{47}$$

Edge loading, when the slab is in an unwarped condition, and when the edges of the slab are warped downward (in the daytime)

$$S_e = \frac{0.572W}{d^2}\left[4\log_{10}\left(\frac{l}{b}\right) + 0.359\right] \tag{48}$$

Edge loading, when the edges of the slab are warped upward (at night)

$$S_e = \frac{0.572W}{d^2}\left[4\log_{10}\left(\frac{l}{b}\right) + \log_{10} b\right] \tag{48a}$$

in which S_c, S_i, and S_e are the maximum unit tensile stresses, in pounds per square inch, in the slab at the corner, in the interior, and in the edge, respectively; W is the load in pounds; and d, a, l, and b have the significance already described. The value of l in each formula is calculated by formula 45. Formulas 47 and 48, as derived by R. D. Bradbury, are merely algebraic reductions of Westergaard's formulas (assuming μ equal to 0.15), and formulas 46 and 48a are empirical modifications of Westergaard's formulas, based upon the results disclosed by the Arlington experimental investigation.

As these formulas involve considerable amounts of labor in their solution, Bradbury suggested a simplified procedure based on the fact that all the formulas have the general form

$$S = \frac{QW}{d^2}$$

in which the value of Q, called a stress coefficient, is controlled by the ratio a/l for stresses induced by corner loadings, and by the ratio

l/b for stresses induced by interior and edge loadings. Values of the stress coefficients in formulas 46, 47, 48, and 48*a* are given in Tables 16, 17, 18, and 19.

E. F. Kelley, of the U. S. Public Roads Administration, has called attention to the fact that it is not possible to assign precise values to the several variables which appear in the stress formulas, in attempting to design a pavement slab for a specific location, but that it is possible to assign general values to these variables which are reasonably conservative; he makes the following recommendations.

1. A range in the value of the subgrade modulus (k) of 50 to 300 lb per cu in. may reasonably be expected in practice, and, for general use, a value of 100 lb per cu in. appears to be appropriate.

2. A value of E_c of 5,000,000 lb per sq in. is representative of concrete in pavement slabs.

3. The impact reaction of a moving vehicle wheel is sufficient to make it a factor in pavement design. Also, the effect of speed on the impact reaction may be neglected for speeds above 50 miles per hour. Also, the magnitude of the impact reaction is definitely related to the frequency of its occurrence, and it is reasonable to assume a frequency of 100 per mile, or that the maximum impact reaction will occur once in about every 50 feet. Also, while the impact reaction is a function of the wheel load, the ratio of the impact reaction to the static load decreases as the wheel load increases. Table 20 contains impact factors and total impact reactions (including the static load) suggested by the U. S. Public Roads Administration for design purposes.

4. The assumption that the contact area of a pneumatic tire on a pavement, resulting from both a static load and an impact reaction, is circular will not lead to a serious error. Figure 91 shows the relation between wheel loads and the radius of equivalent circular areas of tire contacts as developed by the U. S. Public Roads Administration, which is assumed to apply to impact wheel loads as well as static loads.

5. When design is based on dual-tire equipment, it appears reasonable to assume that the *radius* of the area of tire contact for edge loadings is the same as for interior and corner loadings.

6. When design is based on single-tire equipment, owing to the fact that the area of tire contact is undoubtedly changed, it appears desirable to assume that the *area* of the tire contact for edge loadings is the same as for interior and corner loadings. (If r is the radius of a circle, the radius of a semicircle having the same area as the circle is $r\sqrt{2}$.)

TABLE 16

STRESS COEFFICIENTS, Q_c, FOR CORNER LOADS ON CONCRETE PAVEMENTS, CALCULATED BY FORMULA 46

$\mu = 0.15$

(*U. S. Public Roads Administration*)

Ratio a/l	Q_c	Ratio a/l	Q_c	Ratio a/l	Q_c	Ratio a/l	Q_c	Ratio a/l	Q_c	Ratio a/l	Q_c
0	3.000	0.10	2.713	0.20	2.341	0.30	1.928	0.40	1.486	0.50	1.021
0.01	2.982	0.11	2.678	0.21	2.301	0.31	1.885	0.41	1.440	0.51	0.973
0.02	2.958	0.12	2.643	0.22	2.261	0.32	1.841	0.42	1.394	0.52	0.925
0.03	2.932	0.13	2.607	0.23	2.221	0.33	1.798	0.43	1.348	0.53	0.877
0.04	2.904	0.14	2.570	0.24	2.180	0.34	1.754	0.44	1.302	0.54	0.829
0.05	2.875	0.15	2.533	0.25	2.138	0.35	1.710	0.45	1.256	0.55	0.781
0.06	2.845	0.16	2.496	0.26	2.097	0.36	1.666	0.46	1.209	0.56	0.732
0.07	2.813	0.17	2.458	0.27	2.055	0.37	1.621	0.47	1.162	0.57	0.684
0.08	2.780	0.18	2.419	0.28	2.013	0.38	1.576	0.48	1.115	0.58	0.635
0.09	2.747	0.19	2.380	0.29	1.971	0.39	1.531	0.49	1.068	0.59	0.586
										0.60	0.537

TABLE 17

STRESS COEFFICIENTS, Q_i, FOR INTERIOR LOADS ON CONCRETE PAVEMENTS CALCULATED BY FORMULA 47

$\mu = 0.15$

(*U. S. Public Roads Administration*)

Ratio l/b	Q_i	Ratio l/b	Q_i	Ratio l/b	Q_i	Ratio l/b	Q_i	Ratio l/b	Q_i
1.0	0.338	4.0	1.100	7.0	1.407	10.0	1.603	13.0	1.747
1.1	0.391	4.1	1.113	7.1	1.415	10.1	1.609	13.1	1.752
1.2	0.438	4.2	1.127	7.2	1.423	10.2	1.614	13.2	1.756
1.3	0.482	4.3	1.140	7.3	1.430	10.3	1.619	13.3	1.760
1.4	0.523	4.4	1.152	7.4	1.438	10.4	1.625	13.4	1.764
1.5	0.561	4.5	1.164	7.5	1.445	10.5	1.630	13.5	1.768
1.6	0.596	4.6	1.177	7.6	1.452	10.6	1.635	13.6	1.772
1.7	0.630	4.7	1.188	7.7	1.460	10.7	1.640	13.7	1.776
1.8	0.661	4.8	1.200	7.8	1.467	10.8	1.645	13.8	1.780
1.9	0.691	4.9	1.211	7.9	1.474	10.9	1.651	13.9	1.784
2.0	0.719	5.0	1.222	8.0	1.481	11.0	1.656	14.0	1.788
2.1	0.746	5.1	1.233	8.1	1.487	11.1	1.660	14.1	1.792
2.2	0.771	5.2	1.244	8.2	1.494	11.2	1.665	14.2	1.796
2.3	0.796	5.3	1.254	8.3	1.501	11.3	1.670	14.3	1.800
2.4	0.819	5.4	1.265	8.4	1.507	11.4	1.675	14.4	1.803
2.5	0.842	5.5	1.275	8.5	1.514	11.5	1.680	14.5	1.807
2.6	0.863	5.6	1.285	8.6	1.520	11.6	1.685	14.6	1.811
2.7	0.884	5.7	1.294	8.7	1.527	11.7	1.689	14.7	1.815
2.8	0.904	5.8	1.304	8.8	1.533	11.8	1.694	14.8	1.819
2.9	0.923	5.9	1.313	8.9	1.539	11.9	1.699	14.9	1.822
3.0	0.942	6.0	1.323	9.0	1.545	12.0	1.703	....	.....
3.1	0.960	6.1	1.332	9.1	1.551	12.1	1.708	....	.....
3.2	0.977	6.2	1.341	9.2	1.557	12.2	1.712	....	.....
3.3	0.994	6.3	1.349	9.3	1.563	12.3	1.717	....	.....
3.4	1.010	6.4	1.358	9.4	1.569	12.4	1.721	....	.....
3.5	1.026	6.5	1.367	9.5	1.575	12.5	1.726	....	.....
3.6	1.042	6.6	1.375	9.6	1.581	12.6	1.730	....	.....
3.7	1.057	6.7	1.383	9.7	1.586	12.7	1.734	....	.....
3.8	1.072	6.8	1.391	9.8	1.592	12.8	1.739	....	.....
3.9	1.086	6.9	1.399	9.9	1.598	12.9	1.743	....	.....

TABLE 18

STRESS COEFFICIENTS, Q_e, FOR EDGE LOADS ON CONCRETE PAVEMENTS CALCULATED BY FORMULA 48

$\mu = 0.15$

(U. S. Public Roads Administration)

Ratio l/b	Q_e	Ratio l/b	Q_e	Ratio l/b	Q_e	Ratio l/b	Q_e	Ratio l/b	Q_e
1.0	0.205	4.0	1.583	7.0	2.139	10.0	2.493	13.0	2.753
1.1	0.300	4.1	1.607	7.1	2.153	10.1	2.503	13.1	2.761
1.2	0.387	4.2	1.631	7.2	2.167	10.2	2.513	13.2	2.769
1.3	0.466	4.3	1.654	7.3	2.180	10.3	2.522	13.3	2.776
1.4	0.540	4.4	1.677	7.4	2.194	10.4	2.532	13.4	2.784
1.5	0.608	4.5	1.700	7.5	2.207	10.5	2.541	13.5	2.791
1.6	0.672	4.6	1.721	7.6	2.220	10.6	2.551	13.6	2.798
1.7	0.733	4.7	1.743	7.7	2.233	10.7	2.560	13.7	2.806
1.8	0.789	4.8	1.764	7.8	2.246	10.8	2.569	13.8	2.813
1.9	0.843	4.9	1.784	7.9	2.259	10.9	2.578	13.9	2.820
2.0	0.894	5.0	1.804	8.0	2.271	11.0	2.588	14.0	2.827
2.1	0.943	5.1	1.824	8.1	2.284	11.1	2.597	14.1	2.834
2.2	0.989	5.2	1.843	8.2	2.296	11.2	2.605	14.2	2.841
2.3	1.033	5.3	1.862	8.3	2.308	11.3	2.614	14.3	2.848
2.4	1.075	5.4	1.881	8.4	2.320	11.4	2.623	14.4	2.855
2.5	1.116	5.5	1.899	8.5	2.331	11.5	2.632	14.5	2.862
2.6	1.155	5.6	1.917	8.6	2.343	11.6	2.640	14.6	2.869
2.7	1.192	5.7	1.934	8.7	2.355	11.7	2.649	14.7	2.876
2.8	1.228	5.8	1.952	8.8	2.366	11.8	2.657	14.8	2.882
2.9	1.263	5.9	1.969	8.9	2.377	11.9	2.666	14.9	2.889
3.0	1.297	6.0	1.985	9.0	2.388	12.0	2.674	....	.....
3.1	1.329	6.1	2.002	9.1	2.399	12.1	2.682	....	.....
3.2	1.361	6.2	2.018	9.2	2.410	12.2	2.690	....	....
3.3	1.392	6.3	2.034	9.3	2.421	12.3	2.699	. .	.....
3.4	1.421	6.4	2.050	9.4	2.431	12.4	2.707	. .	.....
3.5	1.450	6.5	2.065	9.5	2.442	12.5	2.715	...	.....
3.6	1.478	6.6	2.080	9.6	2.452	12.6	2.722	.	...
3.7	1.505	6.7	2.095	9.7	2.463	12.7	2.730	....	.....
3.8	1.532	6.8	2.110	9.8	2.473	12.8	2.738	. .	.....
3.9	1.557	6.9	2.124	9.9	2.483	12.9	2.746	. ..	.....

TABLE 19

VALUES OF CORRECTION FACTOR, K_e, TO BE ADDED ALGEBRAICALLY TO THE VALUES OF Q_e IN TABLE 18 TO OBTAIN THE EDGE COEFFICIENT, Q'_e, FOR FORMULA 48a

(U. S. Public Roads Administration)

	Values of K_e for Different Values of d in inches								
	d = 4	d = 5	d = 6	d = 7	d = 8	d = 9	d = 10	d = 11	d = 12
0	−0.140	−0.085	−0.040	−0.001	0.032	0.062	0.087	0.111	0.133
0.1	−0.134	−0.079	−0.034	+0.005	0.038	0.067	0.093	0.117	0.139
0.2	−0.117	−0.062	−0.017	0.022	0.055	0.084	0.110	0.134	0.156
0.3	−0.092	−0.037	+0.009	0.047	0.080	0.109	0.135	0.159	0.181
0.4	−0.062	−0.006	0.039	0.077	0.110	0.140	0.166	0.189	0.211
0.5	−0.029	+0.026	0.071	0.110	0.143	0.172	0.198	0.222	0.244
0.6	+0.004	0.059	0.104	0.143	0.176	0.205	0.231	0.255	0.277
0.7	0.036	0.091	0.137	0.175	0.208	0.237	0.263	0.287	0.309
0.8	0.067	0.122	0.167	0.206	0.239	0.268	0.294	0.318	0.339
0.9	0.096	0.151	0.196	0.235	0.268	0.297	0.323	0.347	0.368
1.0	0.123	0.178	0.223	0.262	0.295	0.324	0.350	0.374	0.396
1.1	0.148	0.204	0.249	0.287	0.320	0.350	0.376	0.399	0.421
1.2	0.172	0.227	0.273	0.311	0.344	0.373	0.400	0.423	0.445
1.3	0.194	0.250	0.295	0.333	0.366	0.396	0.422	0.445	0.467
1.4	0.215	0.270	0.316	0.354	0.387	0.416	0.443	0.466	0.488
1.5	0.234	0.290	0.335	0.373	0.407	0.436	0.462	0.486	0.507
1.6	0.253	0.308	0.353	0.392	0.425	0.454	0.480	0.504	0.526
1.7	0.270	0.326	0.371	0.409	0.442	0.471	0.498	0.521	0.543
1.724 *	0.274	0.330	0.375	0.413	0.446	0.475	0.502	0.525	0.547

* When a/d is greater than 1.724, $b = a$, and $K_e = 0.572\,(\log_{10} a - 0.359)$.

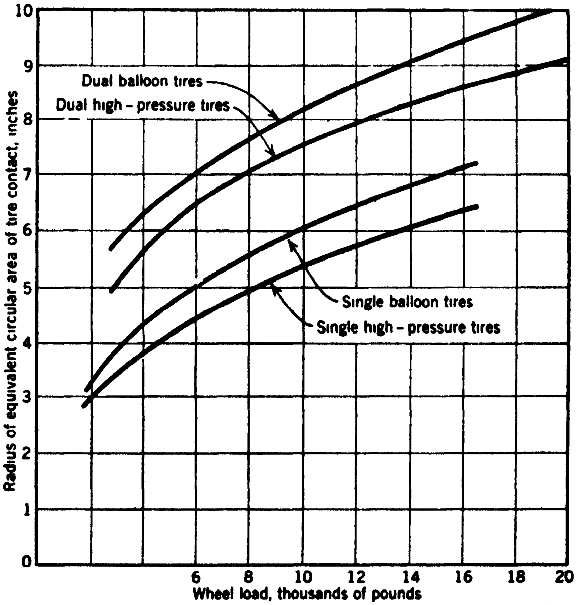
TABLE 20

IMPACT FACTORS AND TOTAL IMPACT REACTIONS OF VEHICLE LOADS ON PAVEMENTS

Speed, 50 mi per hr. Reasonably smooth pavement surface. Frequency of occurrence of maximum impact reaction, 100 per mi.

(*U. S. Public Roads Administration*)

Static Wheel Load, lb	Dual High-Pressure Tires		Dual Balloon Tires	
	Impact Factor	Total Impact Reaction, lb	Impact Factor	Total Impact Reaction, lb
4,000	2 05	8,200	1 70	6,800
5,000	1 80	9,000	1 54	7,700
6,000	1 67	10,000	1 43	8,600
7,000	1 56	10,900	1 37	9,600
8,000	1 48	11,800	1 31	10,500
9,000	1 41	12,700	1 27	11,400
10,000	1 36	13,600	1 24	12,400



Courtesy U. S Public Roads Administration.

FIG. 91. Relation between wheel load (static or impact) and radius of equivalent circular area of tire contact.

7. Inasmuch as the "fatigue limit" or "endurance limit" of concrete subjected to a very large number of rapidly repeated applications of flexural loads is about 55 per cent of the ultimate flexural strength, it has been quite customary to employ about 50 per cent of the assumed flexural strength of the pavement concrete as the allowable unit working stress for the design of the slab cross-section. On most highways the number of vehicles which produce the maximum load stress is a small proportion of the total traffic and the occurrence of such maximum load stresses is not rapid. It probably is safe, therefore, to employ allowable working stresses, in some locations at least, considerably in excess of 50 per cent of the ultimate flexural strength.

The application of the stress formulas and tables will be illustrated by the following example.

Example. Assume a 9000-lb static wheel load on dual balloon tires, and an allowable unit stress of 400 lb per sq in. k is assumed to be 100 lb per cu in. and E to be 5,000,000 lb per sq in.

From Table 20, the total impact reaction, which equals W, is found to be 11,400 lb. From Fig. 91, $a = 8.5$ in. for corner and interior loadings.

The procedure now involves a cut-and-try method because the value of b depends upon the value of d, if a is less than $1.724d$. Values of d will be assumed for each of the three critical load positions and will be adjusted if the variation, one way or the other, from an allowable unit stress of 400 lb per sq in. is significant.

Corner Loading. Assume $d = 8.0$ in. $a/d = 8.5/8.0 = 1.1$. From Table 14, $l = 38.4$ in. $a/l = 8.5/38.4 = 0.22$. From Table 16, $Q_c = 2.26$. Therefore

$$S_c = \frac{Q_cW}{d^2} = \frac{2.26 \times 11{,}400}{64} = 403 \text{ lb per sq in.}$$

Interior Loading. Assume $d = 5.4$ in. From Table 15 (by interpolation), $b = 8.54$ in. From Table 14 (by interpolation), $l = 28.6$ in. $l/b = 28.6/8.54 = 3.35$. From Table 17 (by interpolation), $Q_i = 1.00$. Therefore

$$S_i = \frac{Q_iW}{d^2} = \frac{1.00 \times 11{,}400}{29.16} = 391 \text{ lb per sq in.}$$

Edge Loading. Assume $d = 7.8$ in. $a/d = 8.5/7.8 = 1.1$. From Table 15 (by interpolation), $b = 8.10$ in. From Table 14 (by interpolation), $l = 37.7$ in. $l/b = 37.7/8.1 = 4.65$. From Table 18 (by interpolation), $Q_e - 1.73$. From Table 19 (by interpolation), $K_e = 0.31$. $Q'_e = Q_e + K_e = 1.73 + 0.31 = 2.04$. Therefore

$$S_e = \frac{Q_e'W}{d^2} = \frac{2.04 \times 11{,}400}{60.84} = 382 \text{ lb per sq in.}$$

This value might be considered slightly low in comparison with the allowable unit stress of 400 lb per sq in. Therefore, assume $d = 7.6$ in. $a/d = 8.5/7.6 = 1.1$.

$b = 7.89$ in $l = 37.0$ in $l/b = 37.0/7.89 = 4.7$ $Q_c = 1.74$ $K_c = 0.31$. $Q'_c = 1.74 + 0.31 = 2.05$ Therefore

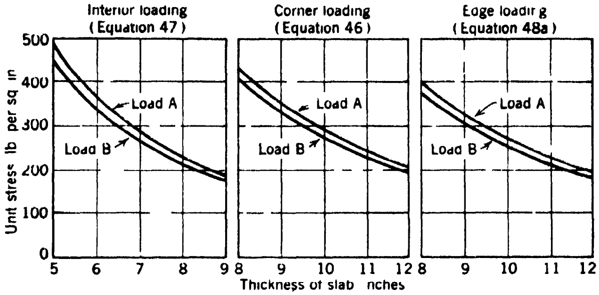
$$S_c = \frac{Q_c'W}{d^2} = \frac{2.05 \times 11{,}400}{57.76} = 405 \text{ lb per sq in}$$

The values of $d = 8.0$ in and 7.6 in for the corner and edge of the slab, respectively, may reasonably be assumed to be the average thicknesses of the slab when a trapezoidal section (as in Fig 88*b*) is employed. Actual edge thicknesses then would be

At the corner	$5.4 + 2(8.0 - 5.4) = 10.6$ in.
At some distance from the corner	$5.4 + 2(7.6 - 5.4) = 9.8$ in

If the slab has a uniform thickness along the edge for some appreciable width as in Fig 88(*a*), the edge thickness then would be 7.6 in

The relation between slab thickness and load stresses may be shown graphically for a number of loadings and, when so developed, will facilitate the determination of slab thicknesses for specific loadings The equivalent load providing for impact may be obtained from Table 20, and the corresponding radius of contact area from Fig 91, for any given static loading This relation for an 8000-lb wheel load on dual high-pressure tires and for a 9000-lb wheel load on dual balloon tires, as developed by the U S Public Roads Administration, is shown in Fig 92



$E_c = 5{,}000{,}000$ lb per sq in $k = 100$ lb per cu in $\mu = 0.15$

Load A-8000 lb Static wheel load - Dual high - pressure tires - Impact reaction 11,800 lb $a = 7.8$ in

Load B-9000 lb Static wheel load - Dual balloon tires - Impact reaction 11,400 lb $a = 8.5$ in

Courtesy U S Public Roads Administration

FIG. 92 Comparison of stresses in a concrete pavement slab caused by 8000-lb wheel load on high-pressure tires and by 9000-lb wheel load on balloon tires

Magnitude of Stresses Caused by Temperature Warping. The warping of concrete pavement slabs due to variations in the temperature of the slabs throughout their depth has been described. Temperature-warping stresses were not considered in the foregoing discussion and may be of sufficient magnitude to affect the design of pavement cross-sections. Westergaard made a theoretical analysis of temperature-warping stresses, and Bradbury developed equations, on the basis of Westergaard's analysis, for calculating temperature-warping stresses along the edge and in the interior of pavement slabs having commonly employed widths and lengths. These equations, as modified by Kelley, are substantially as follows:

Warping stresses along the edge of a slab:

$$S_{xe} = \frac{C_x E_c e_t t}{2} \tag{49}$$

Warping stresses in the interior of a slab

$$S_{xi} = \frac{E_c e_t t}{2}\left(\frac{C_x + \mu C_y}{1 - \mu^2}\right) \tag{50}$$

$$S_{yi} = \frac{E_c e_t t}{2}\left(\frac{C_y + \mu C_x}{1 - \mu^2}\right) \tag{51}$$

in which S_{xe} = the warping stress, in pounds per square inch, in the extreme fiber of the concrete at the edge of the slab, in the direction of the slab length.
S_{xi} = the warping stress, in pounds per square inch, in the extreme fiber of the concrete in the interior of the slab, in the direction of the slab length.
S_{yi} = the warping stress, in pounds per square inch, in the extreme fiber of the concrete in the interior of the slab, in the direction of the slab width
e_t = the thermal coefficient of expansion and contraction of the concrete, per degree Fahrenheit
C_x and C_y = coefficients, C_x in the direction of the slab length and C_y in the direction of the slab width, whose value depends upon the length or width of the slab and upon the radius of relative stiffness of the slab and subgrade, l, in inches.
t = the difference in temperature between the top and bottom of the slab, in degrees Fahrenheit.

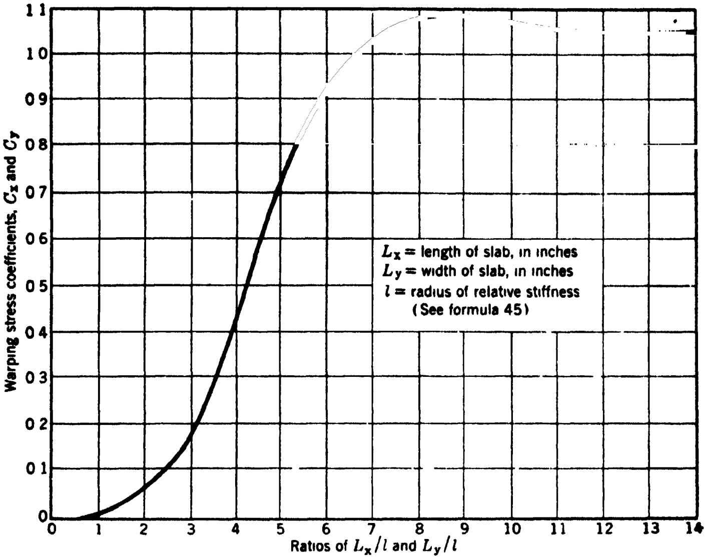
E_c and μ have the same significance as noted previously. Values of C_x and C_y, corresponding to various values of L_x/l and L_y/l, where L_x and L_y are the length and width of a slab, respectively, in inches, are shown graphically in Fig. 93 and also are given in Table 21. Attention is called to the fact that S_{ye}, the warping stress, transversely, in the extreme fiber of the concrete at the edge of the slab, is zero.

In the absence of a mathematical analysis of temperature-warping stresses in the corner areas of slabs, Bradbury has suggested the following approximate formula for the stress on the diagonal line bisecting the corner at the point of maximum load stress:

$$S_{cw} = \frac{E_c e_t t}{3(1 - \mu)}\sqrt{\frac{a}{l}} \tag{52}$$

in which S_{tw} is the temperature-warping stress and the other symbols have the significance already described.

The nature of the warping stress, that is, whether tensile in the top or bottom of the slab, depends upon whether the top of the slab is warmer or colder than the bottom. The U. S. Public Roads Administration adopted the convention that the temperature differential is positive when the top of the slab is warmer than the bottom, and is negative for the opposite condition. The maximum positive differential occurs in the daytime and is greater in summer than in winter; the maximum negative differential occurs during the night and is about the same in summer and winter. (See Table 22) The temperature differential varies greatly with the depth of the slab and, for the Arlington tests, had a maximum positive value of about 4° F per inch of slab thickness. Bradbury concluded that, for design purposes, a maximum positive differential of 3° F per inch of slab thickness was reasonable.



Courtesy R D Bradbury.

FIG. 93. Coefficients for temperature-warping stresses in concrete pavement slabs.

The thermal coefficient of expansion and contraction of concrete in pavements varies from about 0.000 04 to 0.000 07 per degree Fahrenheit, and Kelley has suggested a value of 0.000 005 per degree Fahrenheit for general use.

It is apparent from Fig. 93 that, for slabs shorter than 7l, the length of the slab has a great effect on the magnitude of the longitudinal warping stresses developed. Figure 94, prepared by the U. S. Public Roads Administration, shows graphically values of temperature-warping stresses in the interior, along the edge, and in the corner, of slabs. The effect of the width of slab on transverse

TABLE 21

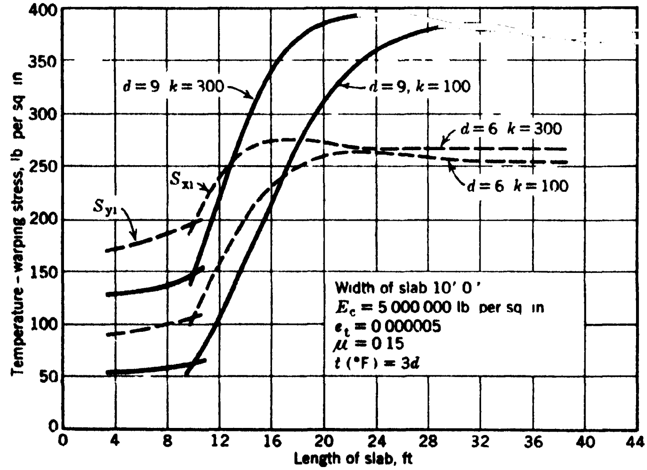
COORDINATES OF DIAGRAM IN FIG. 93

(*U. S. Public Roads Administration*)

$\frac{L_x}{l}$ or $\frac{L_y}{l}$	C_x or C_y	$\frac{L_x}{l}$ or $\frac{L_y}{l}$	C_x or C_y	$\frac{L_x}{l}$ or $\frac{L_y}{l}$	C_x or C_y
1.41	0.010	4.95	0.701	7.78	1.069
2.12	0.051	5.66	0.856	8.49	1.084
2.83	0.148	6.37	0.964	9.90	1.078
3.54	0.309	6.69	1.000	11.31*	1.052
4.24	0.508	7.07	1.032		

* For values of L_x/l or L_y/l greater than 11.31, the values of C_x and C_y are determined by a composite curve constructed as follows:

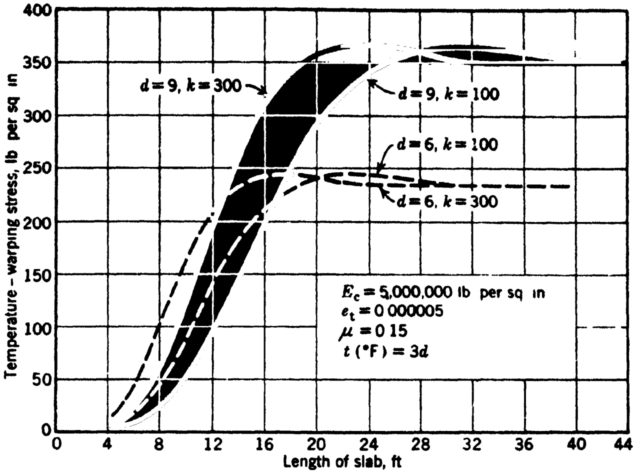
Extend the curve plotted from the data in the above table from ($L_x/l = 11.31$, $C_x = 1.052$) toward ($L_x/l = 14.14$, $C_x = 1.009$) until it intersects a horizontal line drawn through $C_x = 1.043$. C_x or C_y for all values of L_x/l or L_y/l to the right of this intersection is equal to 1.043.



Courtesy R. D. Bradbury and U. S. Public Roads Administration

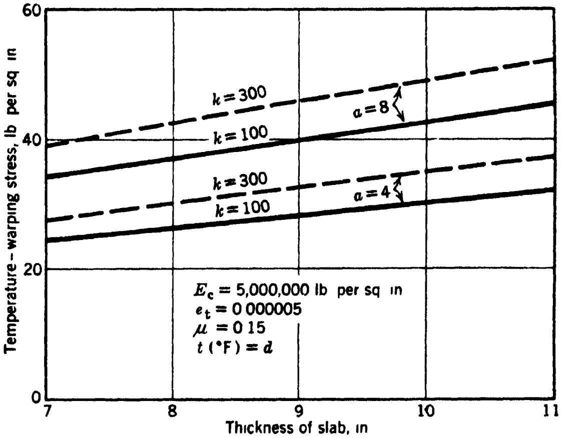
FIG. 94(*a*). Temperature-warping stresses in interior of concrete pavements according to Bradbury's formula.

temperature-warping stresses is shown in Table 23 for slabs 30 ft in length and 10 ft and 20 ft in width, these widths corresponding to



Courtesy R D Bradbury and U S Public Roads Administration.

FIG. 94(*b*). Temperature-warping stresses along edge of concrete pavement slabs according to Bradbury's formula



Courtesy R D Bradbury and U S Public Roads Administration.

FIG. 94(*c*). Temperature-warping stresses in the corner of concrete pavement slabs according to Bradbury's formula

the average single and double traffic lanes, respectively. The significant facts shown in Table 23 are that the warping stresses in a slab of 10-ft width may be as high as 200 lb per sq in. (which is certainly high enough to affect the design of the cross-section), and that the

warping stresses in a slab 20 ft wide may be from 1½ to 3 times as great as in a slab 10 ft wide. The desirability of installing longitudinal joints so that pavement slabs are approximately equivalent in width to a traffic lane, already demonstrated in practice, is thus confirmed by theoretical analysis.

TABLE 22

MAXIMUM TEMPERATURE DIFFERENTIALS IN CONCRETE PAVEMENT SLABS, ARLINGTON, VIRGINIA, TESTS

(*U. S. Public Roads Administration*)

	6-in. Slab				9-in. Slab	
	April to August, inclusive		September to February, inclusive		April to August, inclusive	
	Day	Night	Day	Night	Day	Night
	° F	° F	° F	° F	° F	° F
Maximum	+24 3	−6 5	+15 6	−6 7	+31 0	−9 2
Minimum	+18 7	−4 5	+ 8 2	−1 3	+22 3	−5 7
Average.	+21 2	−5 8	+11 8	−4 1	+26 9	−7 5

TABLE 23

TRANSVERSE TEMPERATURE-WARPING STRESSES IN CONCRETE PAVEMENT SLABS 30 FT LONG

(*U. S. Public Roads Administration*)

$\mu = 0.15$ $E_c = 5{,}000{,}000$ lb per sq in.

$e_t = 0\ 000\ 005$ t (° F.) $= 3d$ (inches)

Subgrade Modulus, *k*, lb per cu in	Width of Slab, ft	Transverse Warping Stress, lb per sq in.		
		Depth of slab, in.		
		6	7	8
100	10	130	120	115
	20	280	320	340
300	10	210	200	190
	20	285	335	380

Combined Stresses Caused by Load and Temperature. The effect of combinations of stresses caused by load and temperature may now be analyzed for each of the three critical positions of load, namely, the corner, the edge, and the interior, of a slab.

Corner. When the slab is curled or warped downward, as in the daytime, the temperature-warping stress in the top of the slab is compressive; at night, the slab warps upward and the temperature-warping stress in the top is tensile. Since a load at a corner produces tensile stress in the top of the slab, the combined stress at night is the sum of the two stresses and is greater than the load stress. The temperature differential at night is smaller than during the daytime (see Table 22) and was assumed as 1° F per in of thickness of slab in plotting the diagrams of Fig. 94(*c*). Since the several variables (i.e., k, a, and d) do not greatly affect the magnitude of the corner warping stress, Kelley concluded that a flat value of 40 lb per sq in. would be sufficiently accurate for practical design purposes.

Edge. At night, the edge of a slab is warped upward, and the effect of the warping is a tensile stress in the top of the slab. The load produces a tensile stress in the bottom of the slab. Thus, the combined stress is less than the load stress During the daytime, the warping produces a tensile stress in the bottom of the slab, and the combined stress is the sum of the load stress and the warping stress. Calculations will show that the latter is the maximum combined stress.[1]

Interior. Kelley reported that the Arlington tests showed that the maximum load stress in the interior of a slab was about the same in the daytime and at night, regardless of the direction of warping of the edges of the slab, and the combined stress therefore is obtained by adding, to the load stress, the warping stress caused by the maximum daytime temperature differential

Total Combined Stresses The total combined stresses (load stress plus temperature stresses) in pavement slabs 10 ft in width, and having lengths of 10, 15, and 30 ft, are shown graphically in Figs. 95 and 96 for various thicknesses of slab and for a value of k (modulus of relative stiffness) $= 100$ It will be observed from Fig 95 that, in slabs 30 ft long, the thickness of the slab does not greatly affect the magnitude of the total combined stress in the edge and in the interior of the slab and that stresses well over 600 lb per sq in. may be expected in slabs of usual thicknesses. Also, if slab lengths of 10 ft are employed, the magnitude of the combined stresses is within limits that, generally, are considered reasonable; this fact is more readily observed from the data in Table 24.

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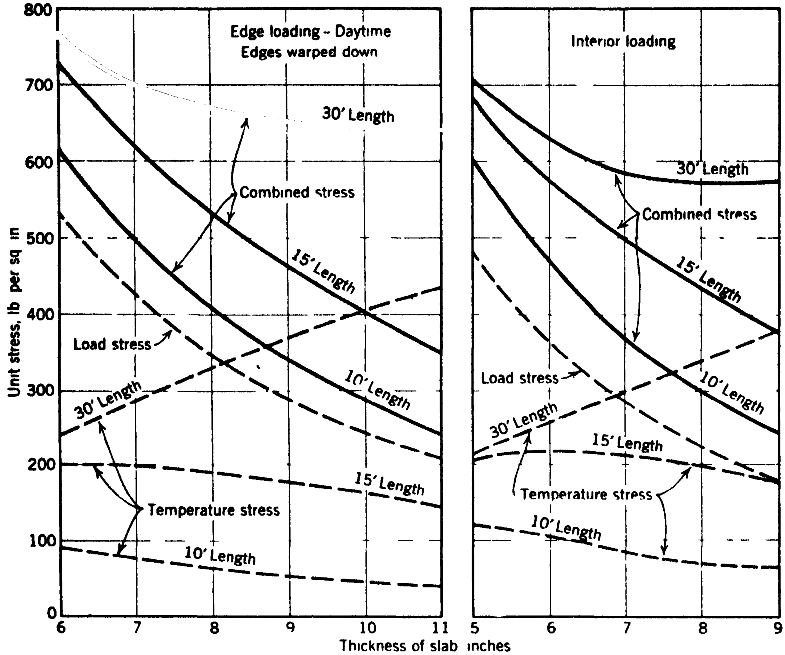
[1] Formula 48 should be employed for calculating the daytime edge-load stress, and formula 48a for calculating the night-time edge-load stress. A temperature differential of 3° F per in. of slab thickness is assumed for calculating the daytime edge-warping stress, and of 1° F per in. of slab thickness for the night-time edge-warping stress.

Determination of Slab Thicknesses. The data in Figs. 95 and 96 and Table 24 show quite conclusively that, for lengths of slabs of 15 ft and greater, the influence of the temperature-warping stresses on the total combined stress is such that the value of a thickened-edge slab in comparison with a slab of uniform thickness is very questionable.



Courtesy U S Public Roads Administration

FIG. 95. Effect of thickness and length on stresses caused by load and temperature warping in edge and interior of a concrete pavement slab, 10 ft wide. (*E. F Kelley, Public Roads, Vol 20, No 5 [July, 1939]*.)

Slabs of Uniform Thickness. Figures 95 and 96 and Table 24 show that, if a slab of uniform thickness is desired, and a unit stress of 400 lb per sq in. is allowable, the slab length should be 10 ft and the slab thickness will be controlled by the corner stress and will be 9 in. The corner, edge, interior, and free-joint-edge stresses will be 390, 330, 250, and 340 lb per sq in., respectively.

If 30-ft slab lengths are selected, reference to Fig. 95 will reveal the fact that it is a practical impossibility to select a slab thickness which will produce a combined unit stress in the edge or in the interior that will have a magnitude as low as 400 lb per sq in. If a 9-in. thickness is selected, the corner, edge, interior, and free-joint-edge stresses will be 390, 650, 570, and 340 lb per sq in, respectively. The edge and interior unit stresses will be so high in proportion to the ultimate strength of the concrete that the slab will inevitably crack; if the resulting crack spacing is reasonably close to 10 ft, the actual combined stresses, as in the first case described, will be about 390 lb per sq in (at an outside corner), and approximately the same result will be obtained as though the slab sections were made 10 ft long to begin with.

TABLE 24

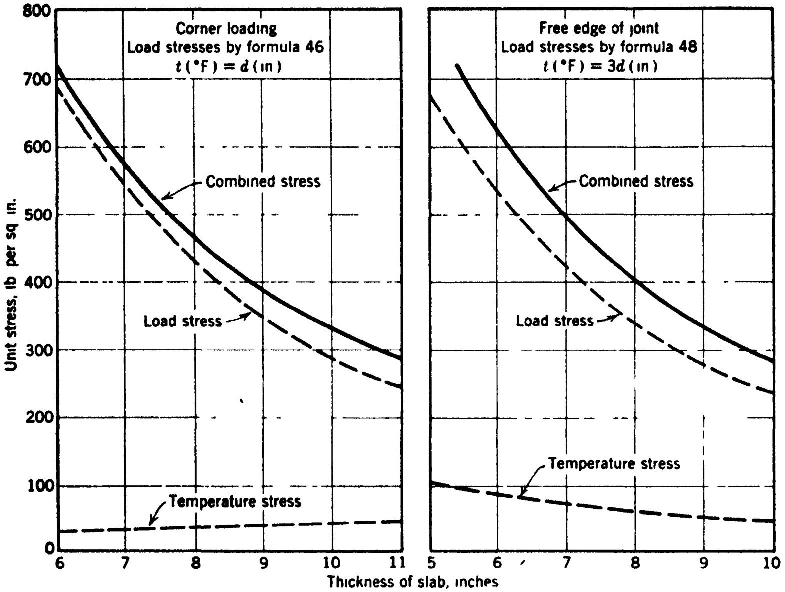
COMBINED STRESSES IN A CONCRETE PAVEMENT SLAB 10 FT WIDE AND OF UNIFORM THICKNESS

(*From Figs. 95 and 96*)

(*U. S. Public Roads Administration*)

Slab Length, ft	Combined Stress, lb per sq in			
	Edge	Interior	Corner	Edge of Free Transverse Joint
Slab 8 in thick				
30	670	570	470	400
15	530	430	470	400
10	400	300	470	400
Slab 9 in thick				
30	650	570	390	340
15	460	380	390	340
10	330	250	390	340

Reference to Table 24 reveals the fact that the corner stresses in pavement slabs of uniform thickness are not greatly in excess of the edge stresses, being 70 and 60 lb per sq in. (18 per cent, each) for the 8-in. and 9-in. thicknesses. Kelley states that "the overstresses[2] that may occur at free transverse joints or at transverse cracks in nonreinforced pavements are so small as to be negligible." However, if the pavements are provided with adequate load transfer devices at transverse joints and, in the case of slab lengths greater than 10 ft, are reinforced so that open cracks will not occur, they may safely be designed for thickness on the basis of the edge stress instead of the corner stress. Thus, for a maximum unit stress of 400 lb per sq in. in a pavement having slab lengths of 10 ft, the required thickness, from Fig. 95, would be 8 in., which is 1 in. less than that for a similar slab designed on the basis of the corner stress.



Courtesy U. S. Public Roads Administration.

FIG. 96. Effect of thickness of slab on stresses caused by load and temperature warping in the corner and edge of a concrete pavement slab, 10 ft. wide, at a free transverse joint. (*E. F. Kelley, Public Roads, Vol. 20, No 5 [July, 1920].*)

Thickened-Edge Slabs. When the length of slab is 10 ft, a thickened-edge slab will result in combined stresses which are reasonably well balanced across the cross-section. However, the determination of the thicknesses of the different portions of the slab presents some difficulties. The Westergaard analyses were developed on the fundamental assumption that the slab is of *uniform* thickness, and they are not applicable to *thickened-edge* slabs. Kelley concluded, however, from data obtained in the Arlington tests, that it was possible to determine, approximately, the stresses to be expected in thickened-edge slabs. The Arlington data showed (1) that the critical stress in the interior of both uniformly thick and thickened-edge slabs was the same from the center of the slab to a point about 2½ ft (or even less in thickened-edge slabs) from the edge; and (2) that the maximum *load* stresses at the edge and in the interior of the slab were approximately the same when the edge thickness was 1.67 times the interior thickness and the edge thickness was reduced to the interior thickness at a uniform rate in a distance of 2 to 2½ ft. See Fig. 88(*b*). It should be borne in mind that the last statement refers to *load stresses* and not to temperature-warping stresses. Kelley made the additional assumption "that the edge-warping stress in a thickened-edge slab is approximately the same as the edge-warping stress in a slab having a uniform thickness equal to the edge thickness of the thickened-edge slab," on the basis of warping stresses measured in the Arlington tests.

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[2] In comparison with the edge stresses.

The approximate unit stresses in the interior and edge of three thickened-edge slabs, calculated on the basis of the three preceding conditions or assumptions, for the same load and other variables as in Figs. 95 and 96, are given in Table 25. Data for slab lengths of 10 ft and 30 ft are included in Table 25.

The determination of the dimensions of cross-sections, as shown in Table 25, follows a "cut-and-try" procedure, and will be described for one case, that of the 9-6-9-in. section having a length of 10 ft. The interior load stress of 370 lb per sq in. and the warping stress of 110 lb per sq in. are the vertical ordinates to the curves marked "load stresses" and "temperature stress, 10′ length," in the right-hand diagram of Fig. 95. The edge stresses are found by first calculating the interior thickness corresponding to an edge thickness of 9 in.: $9 \div 1.67 = 5.4$ in. Referring again to the right-hand diagram of Fig. 95, the load stress of 430 lb per sq in. is the vertical ordinate to the "load-stress" curve for a slab thickness of 5.4 in. The warping stress is found in the left-hand diagram of Fig. 95 and is the vertical ordinate to the "temperature-stress" curve for a 10-ft slab length corresponding to a slab thickness of 9 in.

The foregoing discussion applied to thickened-edge slabs 10 ft long. If a longer slab is to be employed, the maximum unit stresses for the three cross-sections of Table 25 will be so high that cracking may be expected, and, if the spacing between cracks is reasonably close to 10 ft, the same results will be obtained as with 10-ft lengths.

Reference to Table 24 will show that, when slabs of uniform thickness are designed (for thickness) to resist either corner or edge stresses, the edges of free transverse joints or cracks will not require strengthening. The situation is different, however, for thickened-edge slabs. The edges of free transverse joints may be adequately strengthened by thickening the slab or installing suitable load-transfer devices (such as dowels). The edges of transverse cracks may be adequately strengthened by installing steel reinforcing throughout the length of each slab, which will prevent the formation of open cracks.

TABLE 25

COMBINED STRESSES IN THICKENED-EDGE CONCRETE PAVEMENT SLABS 10 FT WIDE *

(E. F. Kelley, U. S. Public Roads Administration, Public Roads, Vol. 20, No. 6, August, 1939)

	Unit Stresses, lb per sq in.				
	Slab Length of 10 ft		Slab Length of 30 ft		
	Interior	Edge	Interior	Edge	Edge of Free Transverse Joint
	9–6–9-in. Section				
Load stress	370	430	370	430	530
Warping stress	110	50	250	370	90
Combined stress	480	480	620	800	620
	10–6.8–10-in. Section				
Load stress	300	370	300	370	440
Warping stress	90	50	290	400	80
Combined stress	390	420	590	770	520
	11.2–7.8–11.2-in. Section				
Load stress	240	310	240	310	360
Warping stress	70	40	330	440	60
Combined stress	310	350	570	750	420

---

* Assumptions with respect to load and other variables are the same as in Figs. 95 and 96.

It is an interesting fact that the ratios of the edge thicknesses to the interior thicknesses of the pavement cross-sections of Table 25 are 1.50, 1.47, and 1.44, for the 9-6-9-in., 10-6.8-10-in., and 11.2-7.8-11.2-in. sections, respectively. In other words, if the interior thickness of a thickened-edge pavement slab (10 ft in length) is determined, on the basis of both load and temperature-warping stresses, the edge thickness will be approximately 1½ times the interior thickness.

The selection of pavement cross-sections generally has been discussed from the viewpoint of maximum legal loads. In some special instances, the critical load may quite properly be less than the maximum legal load. The calculation of slab thicknesses under such conditions by the procedures described may produce thicknesses which will be less than good judgment will approve. A thickness of 5 in. appears to be the minimum which should be employed with concrete.

The diagrams of Figs. 95 and 96 are applicable to wheel loads of 8000 lb on dual high-pressure tires. If larger or smaller loads are employed for design purposes, the determinations of thickness must be obtained directly from the appropriate formulas; a cut-and-try procedure will be necessary, beginning by assuming values for thickness.

The value of great refinement in the determination of slab thicknesses and of the use of a large number of thicknesses is questionable. Edge and center thicknesses are controlled by the height of side forms and by the shape of templates used on subgrading machines, and a large investment in forms and templates would be required of construction organizations if the number of thicknesses were large. The practicable use of a few standard cross-sections is well illustrated by the practice of the Division of Highways of the Illinois Department of Public Works. (See Fig. 97.) Pavements having an interior thickness of 8 in. are used in all cities having a population of over 20,000, and in unincorporated territories in the metropolitan areas of Chicago and East St. Louis; pavements having an interior thickness of 7 in. are used in cities under 20,000 in population having a large amount of industrial hauling and in unincorporated territory adjacent to cities over 20,000 in population other than in the metropolitan areas of Chicago and East St. Louis; pavements having an interior thickness of 6½ in. are used in cities under 20,000 in population with very little industrial hauling and in unincorporated territory not adjacent to cities over 20,000 in population. The laws of Illinois permit an axle load of 16,000 lb, with an increase of 50 per cent in cities of more than 20,000 population.

The necessity of thickened edges on street pavements depends very largely upon the arrangement and design of joints. Figure 98 shows a layout recommended by the Portland Cement Association for street pavements in which a free longitudinal joint is used along the center line of wide streets and doweled longitudinal joints are placed at 10-ft intervals on each side of the center line; thickened edges are required along the free longitudinal center joint. Thickened edges are not required along the outer edges of street pavements when the outside portions of the pavements are used for parking or have integral curbs.

Several typical cross-sections of concrete pavements for both rural roads and municipal streets are shown in Fig. 97.

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CHAPTER XII

CONCRETE PAVEMENTS AND FOUNDATIONS

CONCRETE PAVEMENTS

The first concrete pavement was built in Bellefontaine, Ohio, in 1893, but concrete was not widely used as a paving material until the motor vehicle became an important unit of transportation. Wayne County, Michigan, in 1909, commenced the construction of an extensive county road system of concrete, and the use of concrete for both municipal and rural pavements developed rapidly.

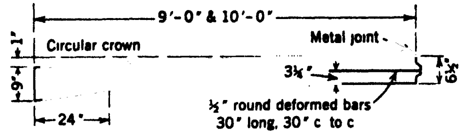
The earliest concrete pavements were crudely designed and constructed in comparison with present standards. Present methods of design and construction are the result of many developments, among which are the extensive studies to determine the laws which control the quality of concrete, the improvement in the quality of Portland cement, studies of the expansion and contraction of concrete pavements, elaborate investigations of the structural action of concrete pavements, the manufacture of greatly improved machinery and tools for construction, and improved methods of producing mineral aggregates for use in concrete.

Design of Concrete Pavements

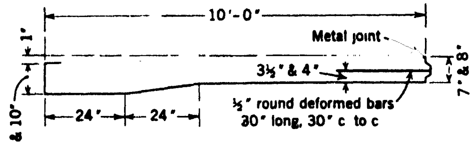
Cross-Sections. The theory of the structural design of concrete pavements was discussed in Chapter XI. The development of Older's theory of stresses in concrete pavements led to the extensive use of thickened edges and longitudinal joints; several typical cross-sections of pavements are shown in Fig. 97. As stated in Chapter XI, the design of cross-sections for street pavements depends upon the arrangement and design of joints. The layout shown in Fig. 98 provides for a free longitudinal joint with thickened pavement edges along the center line of wide pavements, for doweled longitudinal joints along traffic-lane boundaries, and for alternate expansion and contraction joints along the pavement at intervals of 30 ft.

Crowns. The crown of concrete pavements should be as low as possible for rural highways, particularly when the crowned section is retained on superelevated curves. Crowns of $\frac{1}{10}$ to $\frac{1}{8}$ in. per ft are common; crowns on municipal streets are generally slightly higher.

Longitudinal Joints. Longitudinal joints are widely used in concrete pavements for the purpose of reducing transverse warping. It is essential that the adjacent slab sections be tied together and that the joints be effective in transferring loads from one section of the pavement to the adjacent section. The joints are tongue-and-groove, weakened-plane, and butt. The last are generally used only when adjacent slabs are constructed at different times. A metal separating strip forms the tongue-and-groove joint and is held in place by pins, as shown in Fig. 99(*a*); tongue sections are either trapezoidal or triangular. Measurements of stresses in concrete slabs caused by loads near joints, made by the U. S. Public Roads Administration, showed that the tongue-and-groove joint, when dowels bonded to the concrete hold the slab sections together, is most effective for load transfer. Closeness of spacing of dowel bars across longitudinal joints has not been shown to be necessary, and a spacing of 60 in. is common.



Illinois Division of Highways



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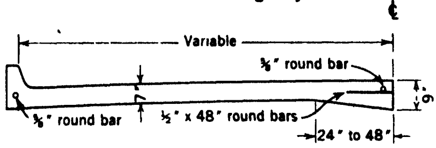
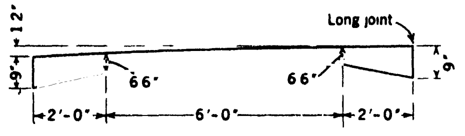


City of Minneapolis



California Division of Highways

Fig. 97. Typical cross-sections of concrete pavements.

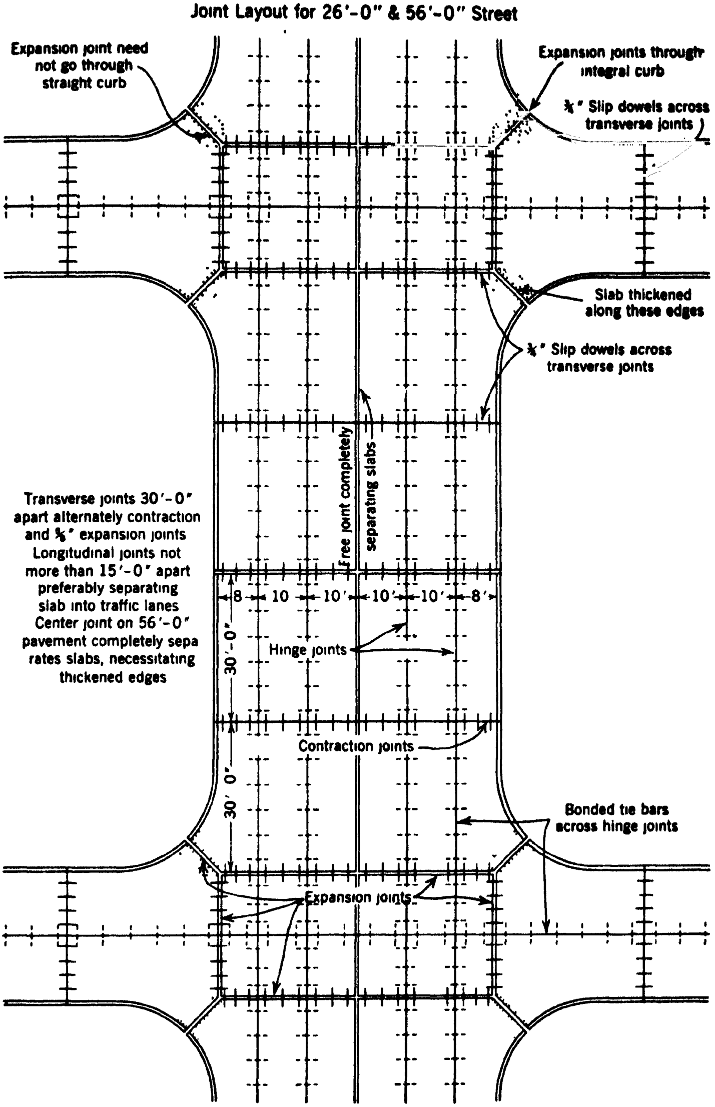
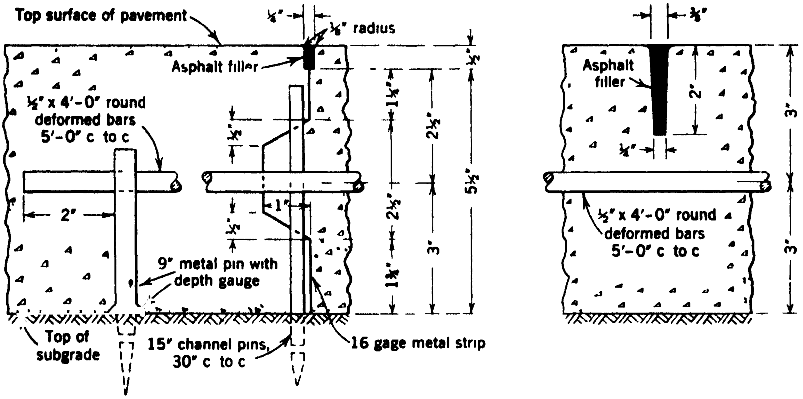

Joint Layout for 26'-0" & 56'-0" Street
Expansion joint need not go through straight curb
Expansion joints through integral curb
¾" Slip dowels across transverse joints
Slab thickened along these edges
¾" Slip dowels across transverse joints
Free joint completely separating slabs
Transverse joints 30'-0" apart alternately contraction and ¾" expansion joints Longitudinal joints not more than 15'-0" apart preferably separating slab into traffic lanes Center joint on 56'-0" pavement completely separates slabs, necessitating thickened edges
8
10
10'
10'
10'
8'
30'-0"
Hinge joints
Contraction joints
30'-0"
Bonded tie bars across hinge joints
Expansion joints

FIG. 98. Layout of joints recommended by the Portland Cement Association for concrete pavements on streets having widths of 26 and 56 ft.

Dowel bars are generally ½ in. in diameter and 4 ft long. Both premolded and poured fillers are used above the metal dividing strip. The upper edges of the groove shown in Fig. 99(*a*) should be slightly rounded to prevent spalling when rotation of the slabs occurs as the result of warping; this rounding of the edges is somewhat more easily done with a poured filler.

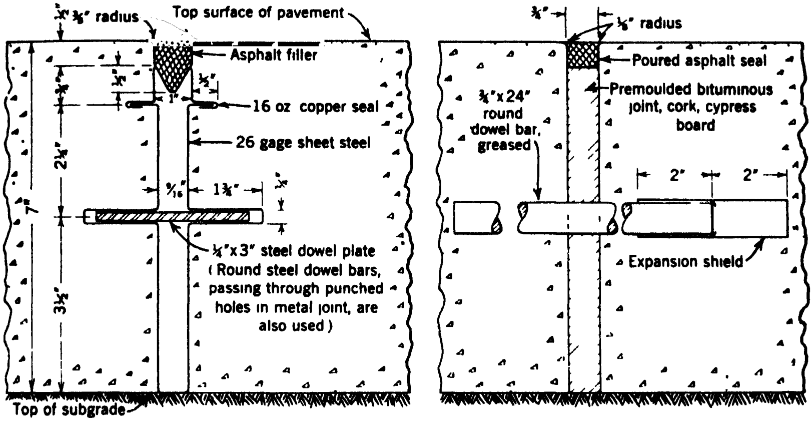
Transverse Expansion Joints. The design and construction of expansion joints involve the selection of a filling material, provision for load transfer, and the avoidance of conditions which will tend to produce high stresses in the slab adjacent to the joint. The three common types of joints may be described as poured bituminous joints, preformed joints, and air-chamber joints The poured bituminous joint is installed by inserting a form in the pavement which is removed after the concrete has hardened and the joint is filled with bituminous material. This joint is about the least expensive type but possesses the disadvantages that the removal of the form, unless performed very carefully, may break the upper portion of the adjacent concrete, and that it is difficult to retain the bituminous material in the joint. Preformed joint fillers (see Fig 100) consist of felt or other fibrous material impregnated with asphalt or tar, rubber, ordinary boards, and cork. Some of these preformed fillers extrude when expansion occurs and may cause a slight bump which is disagreeable to the occupants of vehicles. Some non-extruding, resilient joint materials have been developed which may prove to be satisfactory if their durability over a long period of time is demonstrated. The air-chamber type of joint shown in Fig. 100 must be made of ductile


Top surface of pavement
¼"
⅛" radius
Asphalt filler
½" x 4'-0" round deformed bars 5'-0" c to c
2"
½"
1"
½"
1¼"
2½"
2½"
1¾"
½"
5½"
3"
9" metal pin with depth gauge
Top of subgrade
15" channel pins, 30" c to c
16 gage metal strip
⅜"
Asphalt filler
2"
¼"
3"
3"
½" x 4'-0" round deformed bars 5'-0" c to c

(*a*) Tongue and Groove Joint (*b*) Weakened-plane Joint

FIG. 99 Longitudinal joints for concrete pavements

metal, such as copper, which will not crack or split when expansion and contraction occur. Load transfer at expansion joints is generally secured by means of round steel dowel bars which extend into the slab on each side of the joint. These dowels must be free to move when expansion and contraction occur; otherwise, contraction will cause the pavement to crack near the ends of the dowels and expansion will cause crushing and fracture of the concrete adjacent to the joint. This freedom of movement is secured by painting and greasing the dowel bars to prevent bond with the concrete and by so placing short metal sleeves at one or both ends that end movement of the dowels is possible Diameters of dowel bars vary from ½ to ¾ in. Studies of load transfer by the U. S. Public Roads Administration show that dowel bars should be closely spaced to be effective, and a spacing of 12 to 15 in. now is generally considered necessary. Dowel-plate joints are sometimes used and are effective for load transfer. The problem of load transfer across transverse joints is intimately related to longitudinal warping. If transverse joints were completely effective so that 50 per cent of the load on the slab at one side of the joint were transferred to the adjacent slab, longitudinal warping would be restrained by the rigid joints and no benefit would be received owing to a reduction of stresses induced by the restrained warping. It is obvious that dowel bars must be installed carefully so that they are parallel to the longitudinal axis of the pavement and to the surface of the pavement so that free movement is secured when


¼"
⅜" radius
Top surface of pavement
Asphalt filler
¼"
½"
1"
16 oz copper seal
26 gage sheet steel
2¾"
7"
9/16"
1¾"
¼"
¼"x3" steel dowel plate (Round steel dowel bars, passing through punched holes in metal joint, are also used)
3½"
Top of subgrade
¾"
⅜" radius
Poured asphalt seal
Premoulded bituminous joint, cork, cypress board
¾"x24" round dowel bar, greased
2"
2"
Expansion shield

(*a*) Air-chamber Joint (*b*) Preformed Joint

FIG 100 Expansion joints for concrete pavements

the pavement expands and contracts. Dowel bar assemblies which may be set in place as a unit are therefore desirable.

Transverse Contraction Joints. Contraction joints formerly were omitted between expansion joints, and transverse contraction cracks therefore occurred after the concrete slab was placed, sometimes within very short intervals of time. Dependence was placed on reinforcement to prevent wide cracks from forming. This practice had some disadvantages. Contraction cracks often are irregular and therefore somewhat unsightly. The square edges of slabs at contrac-

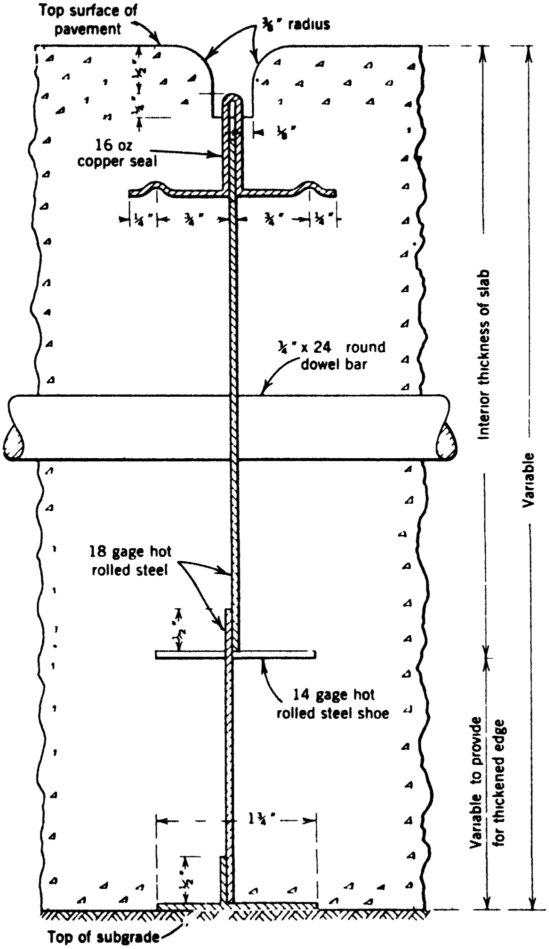


Fig 101. Sealed-metal transverse contraction joint for concrete pavements Used by Division of Highways, Illinois Department of Public Works.

tion cracks tend to spall because of the wedging action which takes place as the result of longitudinal warping. Irregular longitudinal contraction sometimes produces secondary contraction cracks and occasionally diagonal cracking at corners due to the reinforcement being bonded to the concrete. Transverse contraction joints, of the weakened-plane and sealed metal types, are very extensively used. The weakened-plane contraction joint is constructed in the same way as the weakened-plane longitudinal joint; the sealed metal joint is illustrated in Fig. 101.

Most designers favor the use of dowel bars, with sleeves, across contraction joints. Measurements of load transfer at contraction joints, made by the U. S. Public Roads Administration, indicate that dowel bars are desirable. The details of size, spacing, and installation are the same as for expansion joints.

Transverse Warping Joints. The desirability of installing joints to permit comparatively free longitudinal warping was demonstrated by the investigation of pavement stresses conducted by the U. S. Public Roads Administration. The results of this investigation have not yet been reflected in design procedure. Joints similar to the weakened-plane, transverse contraction joints have been recommended for warping joints.

Steel Reinforcement. Properly designed joints in concrete pavements appear to eliminate the necessity of providing additional steel for reinforcement. If transverse contraction joints are omitted, steel reinforcement is desirable because it prevents contraction cracks from opening up.

Steel reinforcement usually is in the form of welded wire fabric or bar mats assembled by tying together transverse and longitudinal bars at points of intersection. The principal advantage of bar mats is their ease of placing. Welded wire fabric, when shipped in flat sheet form, however, is not exceedingly difficult to handle.

MATERIALS FOR CONCRETE PAVEMENTS

Fine Aggregate. Sand is most widely used as the fine aggregate for concrete pavements. Crushed-stone screenings have been employed to only a limited extent. Some pavements in which limestone screenings served as fine aggregate developed surface scaling, and it is largely for this reason that screenings have not been very widely used. Sand should be composed of hard, durable particles and should be free from organic matter or foreign substances. A clay and silt content of not more than 2 per cent is to be preferred on account of the tendency of the clay to rise to the surface of the pavement during finishing operations and cause weakening and subsequent scaling of the concrete. Satisfactory concrete may be obtained with sands having a wide range in grading. The following grading requirements are typical:

	Per Cent
Passing ⅜-in. sieve	100
Passing No. 4 sieve	95 to 100
Passing No. 16 sieve	35 to 75
Passing No. 50 sieve	8 to 25
Passing No. 100 sieve	0 to 5
Loss by washing	0 to 2

Coarse Aggregates. Screened gravel, crushed gravel and crushed boulders, crushed slag, and crushed stone are used to the greatest extent for coarse aggregate for concrete pavements.

The desirable characteristics of coarse aggregates are cleanness, reasonable hardness and structural strength, round or angular shape, and a satisfactory size and grading of particles.

The cleanness and freedom from deleterious substances are controlled by specifying maximum percentages, the following requirements being representative of good practice:

	Per Cent
Removed by decantation	1
Shale	1
Coal	1
Clay lumps	0.5
Soft fragments	3
Thin, elongated, or laminated particles	3
Total deleterious substances	5

It is rather difficult to lay down a definite rule for the hardness or structural strength of coarse aggregates. A maximum of 6 to 7 per cent of wear in the Deval abrasion test is frequently specified for crushed stone. The Los Angeles abrasion testing machine may supplant the Deval abrasion machine. An abrasion test requirement (conducted in the Deval abrasion machine) is sometimes adopted for gravel but has not been very satisfactory. Crushed blast-furnace slag is usually required to have a weight of 70 to 75 lb per cu ft.

Coarse aggregate particles should be rounded or angular in shape. Flat or slivery particles are objectionable because they may break easily if near the surface, or may cause laminations or planes of weakness in the concrete.

A wide variety of sizes of coarse aggregates are used for concrete pavements. In general, the coarser materials are desirable because, for the same strength, they require less cement, which is the costliest ingredient of concrete. The cost of the concrete is the determining factor in the selection of the coarse aggregate, and in some locations aggregates that are available must be used rather than more expensive, though more desirable, materials. A maximum size of 2½ to 3 in. is most widely favored if it is economically available. When a coarse aggregate having a maximum size of more than 1½ in. is to be used, it is desirable to employ two or more sizes in order to obtain greater uniformity of the concrete mixture.

Water. Most specifications for concrete pavements provide that water for the concrete mixture shall be clean and free from acids or alkalis. Generally, water from any freely running supply will be satisfactory. Water from muddy streams or from stagnant pools may be objectionable.

Cement. Standard Portland cement is used for almost all concrete-pavement construction. In special cases where it is inadvisable to keep a street or road closed for the time required for proper curing, high-early-strength Portland cement is used; examples are street and road intersections and heavily traveled thoroughfares where detour conditions are unsatisfactory.

Admixtures. Calcium chloride has been used to some extent for accelerating the hardening of concrete pavements, but its action varies somewhat with different Portland cements; 2 to 4 per cent, by weight of the cement, is generally used, dissolved in the mixing water.

Composition of the Concrete. The quality of pavement concrete is usually measured in terms of its compressive strength or flexural strength, or both. A minimum compressive strength of 3500 lb per sq in., and a minimum flexural strength of 650 lb per sq in. at the age of 14 days, for specimens of concrete taken on the work and cured under carefully controlled conditions, are representative of good practice. Several methods employed in specifications to control the quality of the concrete include the use of (1) definite proportions of the materials or a fixed cement content, (2) a minimum limitation on strength or a maximum limitation on the water-cement ratio, and (3) a combination of the foregoing procedures. The first method is possible only when the size and grading of the mixed aggregate are carefully controlled. Table 26 shows limitations for the composition of concrete for several kinds and sizes of coarse aggregate: in this specification no strength requirement is stipulated and it is assumed that satisfactory strengths will be obtained by the control of proportions, water content, and cement content. Another type of specification, which is based on proportioning by the "mortar-voids theory,"[1] is employed by some highway organizations and is illustrated by Michigan practice (*1940 Standard Specifications*), which provides for two sizes of coarse aggregate, the largest having a maximum size of 2 to 2¼ in., and has the following limitations:

Approximate proportions, by volume	1 : 2¼ : 4
Compressive strength at 28 days, lb per sq in., minimum	2500
Modulus of rupture, lb per sq in., minimum	
At 7 days	550
At 28 days	650
Estimated cement content, sacks per cubic yard of concrete	5.5

The Michigan specification provides that "The concrete mixture will be designed by the Engineer by the mortar-voids theory for the materials to be used. The design will give the amounts of water, cement, fine and coarse aggregate required per cubic yard of concrete."

TABLE 26

COMPOSITION OF CONCRETE FOR CONCRETE PAVEMENTS

(*Louisiana Department of Highways, 1940 Standard Specifications*)

Coarse Aggregate		Approximate Proportions, by Volume	Maximum Water Content, gal per sack of cement	Minimum Cement Content,* bbl per cu yd of concrete
Size	Material			
1½ in. to No. 4	Gravel (1 size)	1 : 2 : 3	5.50	1.45
2¼ in. to No. 4	Gravel, stone, or both (2 sizes)	1 : 2 : 3¾	5.75	1.30
2 in. to No. 4	Gravel (1 size)	1 : 2 : 3½	5.50	1.35
2½ in. to No. 4	Gravel, stone, or both (2 sizes)	1 : 2 : 4.6	6.00	1.25
2½ in. to No. 4	Stone (1 size)	1 : 2 : 3½	5.50	1.50

* Sum of parts of aggregate governs if cement content is higher than above figures.

The workability of the concrete is important and depends upon the method of finishing the pavement. The slump test is frequently specified for determining workability. Slumps of 2 to 3 in., 1 to 2 in., and ½ to ¾ in. are suitable for hand finishing, machine finishing, and vibration, respectively.

Control of Concrete Mixtures. The quality of concrete mixtures is controlled by tests of the materials and of the hardened concrete.

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[1] *Bulletin* 137, Engineering Exp. Sta., Univ. of Ill. (October, 1923), "The Strength of Concrete," by Arthur N. Talbot and Frank E. Richart.

Frequent determinations of the moisture content of fine aggregates are essential in controlling the proportions of fine aggregate and water. Strength tests on fabricated specimens are desirable as a check on the quality of concrete, and 3-day or 7-day tests have some value for control purposes. Flexural strength tests may be made on the job by means of comparatively simple apparatus, but compressive strength tests usually must be conducted in adequately equipped laboratories.

Compressive strength tests are sometimes performed on specimens drilled from hardened pavements. Such tests have little or no value for control purposes but do have statistical value. Drilled specimens are very valuable for determining the actual thickness of pavements.

Construction of Concrete Pavements

Concrete pavements are constructed by the wet-batch method, in which central mixing plants or truck mixers are used, and by the dry-batch method, in which the materials are measured at a central proportioning plant and hauled to a paving mixer which mixes the materials and deposits the concrete on the subgrade. The wet-batch method, particularly with truck mixers, is satisfactory but is practiced to a much greater extent on municipal street paving than on rural road construction.

Storage and Handling of Aggregates. The usual equipment for measuring aggregates consists of volumetric or weighing containers which are filled from overhead bins by gravity. Cranes, mounted on crawler treads, handle the aggregates from cars or stock piles to the bins.

Some storage of aggregates is necessary so that construction operations will not be stopped by delays in shipments or breakdowns of producing plants. At the same time the contractor saves money when he transfers aggregates direct from railroad cars to the measuring plant.

Care must be taken in placing coarse aggregates in stock piles to prevent segregation of sizes. Segregation is greatly reduced when two sizes of coarse aggregate are used, but it is desirable to build up storage piles in wide successive layers rather than in a conical pile. Sand seldom segregates into different sizes because it usually contains moisture which holds the particles together.

Handling of Cement. Cement in sacks is handled most economically when it may be unloaded direct from cars to trucks supplying the mixer Some storage is desirable in order to prevent shutdowns due to delays in shipment, and weather-tight sheds should be provided.

Shipment of cement in bulk has increased in recent years as the result of adopting weighing methods of measurement and because of improved methods of handling. Hopper-bottom cars and motor trucks are used when conditions are favorable, but portable pumps for box-car shipments and containers which are loaded on flat cars and unloaded into storage bins by means of cranes are more adaptable to most conditions. The cement is discharged from the storage bins to weighing batchers by means of screw or belt conveyors.

Water Supply. As the construction of a concrete pavement requires large quantities of water for mixing and curing, a dependable source of supply is imperative. Municipal water supplies can be utilized for street paving and for paving on rural roads located near cities and villages. The water for most rural road projects must be obtained from nearby lakes or streams by means of portable pumps. Booster pumps may be necessary on long pipe lines. A 3-in., heavy-duty pump, having a capacity of 15,000 gal per hr, is the common type. A 3-in. pipe line is necessary to supply the required amount of water for a 27-cu-ft mixer and for curing, and should be equipped with tees for hose connections to the mixer and for sprinkling, relief pressure valves, unions, and gate valves.

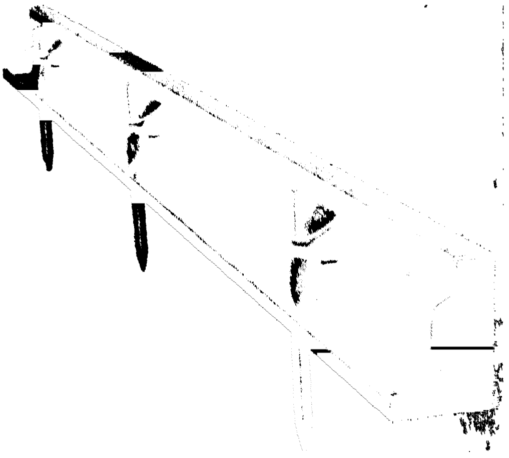
Preparation of Subgrade. Subgrades for concrete pavements are usually shaped to a flat or slightly crowned cross-section during grading and to the final cross-section after the side forms have been set in place.

It is essential that the subgrade provide uniform as well as firm support for the pavement slab, and scarifying and rolling are therefore practiced with most soils to eliminate non-uniformity and to consolidate the soil. The final cutting to grade generally should be performed after rolling to insure having a true cross-section. The cross-section ahead of the mixer should be checked by means of a scratch template, consisting of a heavy frame which rides on the side forms and which carries spikes or teeth so arranged that their lower ends are set at the correct elevation. Some highway departments require that a scratch template also be used behind the mixer.

Dry subgrades absorb water from freshly deposited concrete and cause shrinkage cracks; subgrades therefore should be sprinkled before the concrete is placed. Sprinkling should be done six to eighteen hours in advance of placing the concrete and should be continued until the upper soil is quite wet but not muddy. Light sprinkling at the time of placing the concrete often is desirable.

Forms. Heavy steel side forms, having wide bottom faces, are used for retaining the concrete and as side rails for the operation of finishing machines, and are held in place by long, heavy steel pins which are securely attached to the forms and are driven into the subgrade. When integral curbs are to be constructed, the pavement is usually struck off and finished to the normal cross-section without curbs; special curb forms are then attached to the tops of the standard side forms, and the concrete for the curbs is placed and finished while the concrete below is still plastic.

Placing Reinforcement. Bar mat reinforcement for concrete pavements is best held in place by means of chairs, consisting of bent bars, which rest on the subgrade and on which the mats are placed. This method of placing is particulaly adaptable when the mixer travels on the shoulder but tends to slow up the work when the mixer travels on the subgrade between the forms. Wire mesh reinforcement is placed most effectively by using a preliminary strike board, which strikes off the concrete to the required depth of the reinforcement below the surface of the pavement, and then laying the reinforcement in place. The mixer then deposits another layer of concrete above the reinforcement. The preliminary strike board may be moved along the forms by hand, by means of a winch on the mixer, or by the finishing machine.



Courtesy of Heltzel Steel Form Co

FIG. 102 Typical steel form for concrete pavement

Construction of Longitudinal Joints. Tongue-and-groove joints are formed by means of the metal separating strip shown in Fig. 99 (*a*), which is held in place by steel bars or pins driven into the subgrade through punched holes in the strip. Dowel bars are placed through punched holes in the strip and are held in place by means of pins driven into the subgrade. Placing bars or templates are useful for putting the metal separating strips in the correct position.

Weakened-plane joints may be formed by hand, but a more satisfactory method consists of cutting a groove in the concrete with a metal disc mounted on a rigid frame which rides on wheels on the side forms. The groove is then rounded with an edging tool operated by hand from a bridge which rides on the forms.

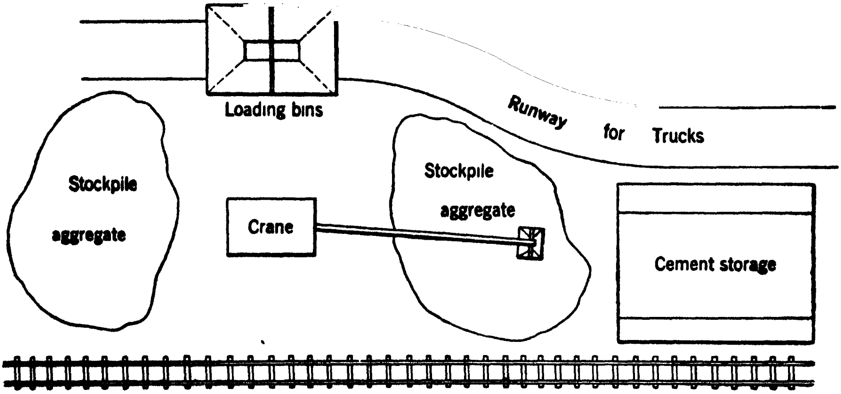
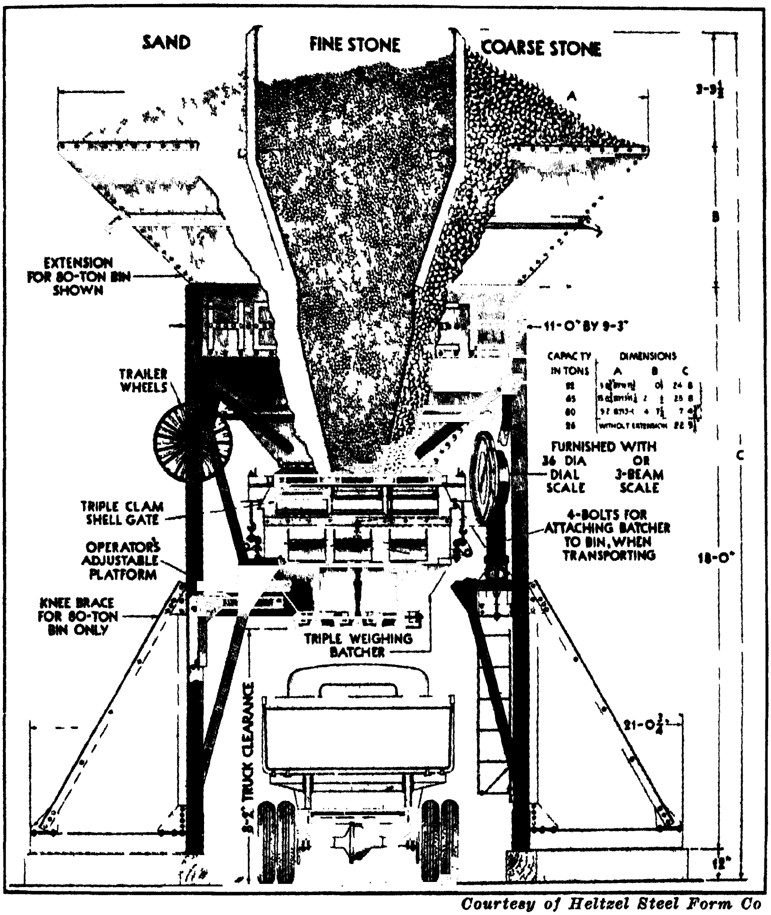


FIG. 103. Typical layout of a central proportioning plant for handling the materials for the construction of a concrete pavement. The cement is generally unloaded directly from railroad cars to the trucks carrying the aggregates, and the cement storage house is used only for a reserve supply to prevent shutdowns caused by delayed shipments

Construction of Transverse Joints. Transverse joints include expansion joints, contraction joints, and construction joints.

The installation of all transverse joints is a very important detail of the construction of concrete pavements. The joint must be placed accurately at right angles to the longitudinal axis of the pavement to prevent lateral movement and vertically so that expansion will not tend to cause heaving Dowels must be parallel to the surface, and to the longitudinal axis, of the pavement, so that breakage will not be caused by bending of the dowels during the expansion and contraction of the pavement. The use of dowel bar assemblies facilitates the accurate installation of all transverse joints.

Preformed expansion joint material is placed by means of a special bulkhead which holds it in place until the concrete has been deposited and struck off on both sides, when it is removed and the concrete finished. A metal placing bulkhead is best for this purpose because it usually may be withdrawn more easily than wood and leaves a smaller space to be filled with concrete A metal bulkhead in the form of a deep channel, slotted to permit withdrawal at dowels and notched at the bottom to allow the concrete to hold both sides of the preformed


SAND
FINE STONE
COARSE STONE
A
B
C
EXTENSION FOR 80-TON BIN SHOWN
11-0" BY 9-3"
CAPACITY IN TONS
DIMENSIONS
A B C
WITHOUT EXTENSION
TRAILER WHEELS
FURNISHED WITH 36 DIA DIAL SCALE OR 3-BEAM SCALE
TRIPLE CLAM SHELL GATE
4-BOLTS FOR ATTACHING BATCHER TO BIN, WHEN TRANSPORTING
OPERATOR'S ADJUSTABLE PLATFORM
18-0"
KNEE BRACE FOR 80-TON BIN ONLY
TRIPLE WEIGHING BATCHER
TRUCK CLEARANCE
21-0½"
12"

Courtesy of Heltzel Steel Form Co

Fig 104 Diagram of plant for weighing aggregates for concrete

filler as the bulkhead is removed, has been widely used The joint is submerged to a depth of ½ in below the top of the pavement to permit finishing operations to be conducted

Cypress boards do not require a special bulkhead but are held in place by means of T-shaped metal pins which are withdrawn when the space above the boards is grooved to receive the asphalt filler.

The poured type of bituminous joint is constructed by staking in place one or more wooden or metal plates, which are slightly tapered to facilitate removal, and which are usually removed on the morning after the placing of the concrete.

Measurement of Materials. The accurate measurement of all materials is essential in order to secure the desired quality and uniformity of quality of the concrete. Aggregates always should be weighed on account of the great inaccuracy of volumetric measurement, and satisfactory proportioning plants therefore are equipped with bins which discharge into weighing hoppers. Sometimes there is a separate hopper for each aggregate, but a single hopper with separate compartments for each aggregate also is used, the weight of each aggregate being determined by a separate beam scale or accumulating on a dial scale. The ease with which weights may be checked is a decided advantage of dial scales Some beam scales are equipped with "telltale" dials for showing the last 200 to 300 lb of each specified load and the overload, if any.

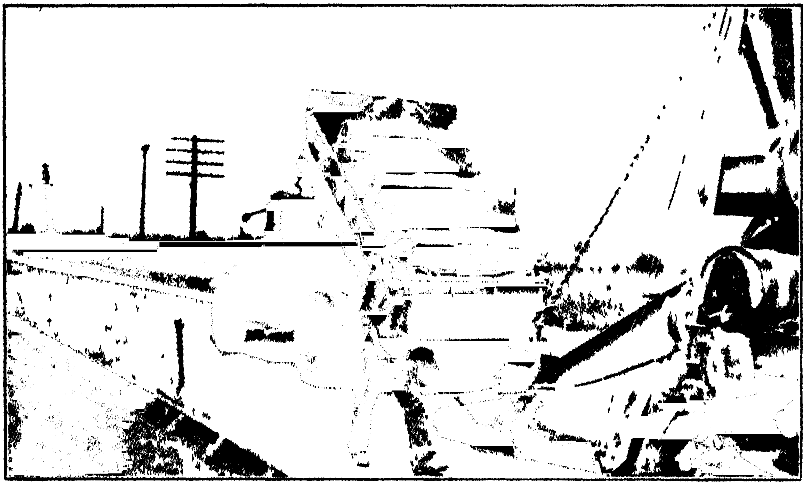


FIG. 105. Multiple-batch truck delivering aggregates and cement to a paving mixer.

Cement is measured in full sacks, except in bulk shipments, and is added to the batch at the car or cement storage house. When cement is shipped in bulk, a separate cement bin and weighing hopper are employed.

The water is usually measured in gallons, although some mixers are equipped for weighing the water. The free moisture content of each aggregate must be determined and taken into consideration in establishing the quantities of water and aggregates for each batch.

Hauling Equipment. Motor trucks equipped with dump bodies having separate compartments are widely used for hauling materials from central proportioning plants. Small motor trucks and trailers, equipped with batch boxes which are unloaded into the mixer skip by means of a crane mounted on the mixer, are preferred by some contractors because they may be operated on the shoulders to avoid disturbing the subgrade. When trucks discharge directly into the skip of the paving mixer, it is desirable, on many soils, to use turntables to prevent excessive marking of the subgrade.

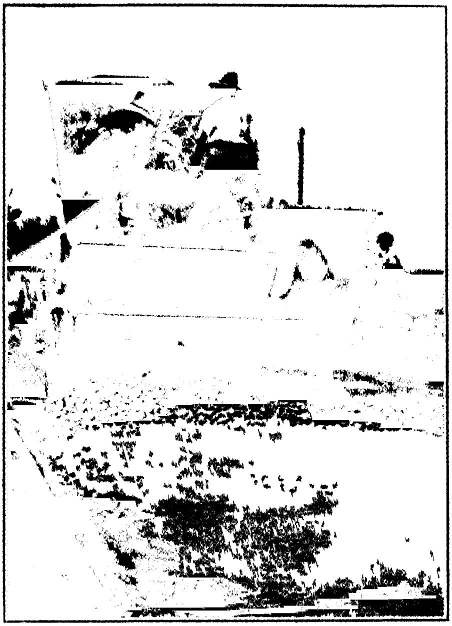


FIG 106 The result of hauling mixed concrete several miles in a flat-bottom truck body.

Central mixing plants forming an integral part of the construction equipment, once used to a limited extent on concrete paving, now are used infrequently The principal disadvantage of central mixing plants is the necessity of providing special hauling equipment on account of the segregation of the ingredients of concrete hauled in ordinary truck bodies (see Fig 106) Commercial central mixing plants, however, are often used, particularly for street paving; truck mixers are used for hauling the concrete which is partially mixed at the central plants, mixing being completed while en route to the job.

Mixing the Concrete. Paving mixers which are operated on the subgrade are self-propelled and of the boom-and-bucket type; the concrete is discharged into a bucket which moves along a boom that may be swung over the width of the pavement between the forms. The aggregates, cement, and water should all be introduced into the mixer at the same time in order to obtain a uniform consistency and in order that the least amount of time will be consumed in charging the mixer. Suitable mixers have a water-measuring tank which is filled from an auxiliary tank and accurately measures the water when the mixer is operating either on a level or sloping subgrade. A mixing time of 60 seconds is practically universal practice, and most highway departments require mixers to be equipped with automatic timing devices. Various sizes of mixers have been used for paving, but a mixer of 1-cu-yd capacity per batch has been found by experience to be the most efficient size for most conditions. Dual drum mixers, in which the concrete is partially mixed in one drum and transferred to a second drum for completion of mixing, have a limited application and possess the advantage of having a large output.

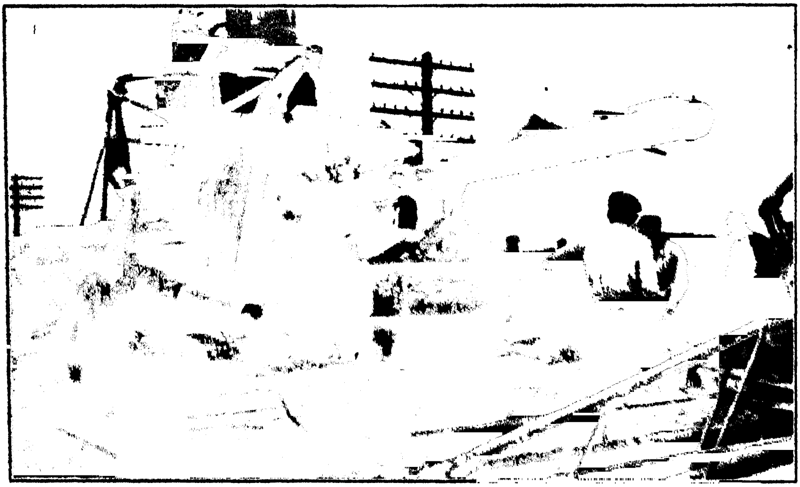


FIG. 107. Placing concrete with boom-and-bucket type of paving mixer. Note scratch template hauled behind mixer.

Mixers of 7- to 10-cu-ft capacity per batch frequently are used for mixing the concrete for widening on curves, for curbs, and for other special conditions. Ordinarily these auxiliary mixers are not equipped with boom-and-bucket delivery.

Deposition of Concrete. The concrete should be deposited on the subgrade so as to avoid as much handling as possible and so that segregation of the materials will be prevented. The boom-and-bucket type of paving mixer permits the control of deposition by the swing of the boom and the travel of the bucket along the boom and has entirely displaced the chute type of mixer which tends to cause segregation and makes placing more difficult. Some hand spreading generally is necessary ahead of the finishing machine; this should be done by shovels, as hoes or rakes tend to cause segregation of the mortar and coarse aggregate. Ready-mixed concrete may be deposited directly on the subgrade by operating the trucks between the forms or on the shoulders. The latter method of depositing is preferable with steel reinforcement or if the subgrade cuts easily under the truck wheels and is particularly applicable when the pavement is constructed in single lanes

Fig 108 Finishing machine spreading and smoothing concrete.

Striking Off and Finishing the Concrete. After the concrete has been deposited on the subgrade, it is struck off with a strike board or screed which rides on the side forms and is shaped to the required crown. Hand-operated strike boards have in general been superseded by power-operated finishing machines equipped with strike boards. Single-screed machines are used, but a double-screed machine, in which the first screed is equipped with a strike board (and is set slightly high) and the second screed, set back of the first screed, has a wide face which compresses and smooths the concrete, is a more satisfactory device Tampers, mounted behind the strike-off screed, formerly were widely used, but their action tended to bring the mortar to the surface of the pavement and cause scaling due to unequal expansion and contraction, and they now are not so extensively used Surface vibrators are used, however, mounted between the two screeds on finishing machines, the concrete is struck off high by the strike board, compacted by the vibrator, and smoothed by the second screed Some concrete always should be carried along ahead of the strike board but should not pile up so high that it will run over the strike board or be carried outside of the edges

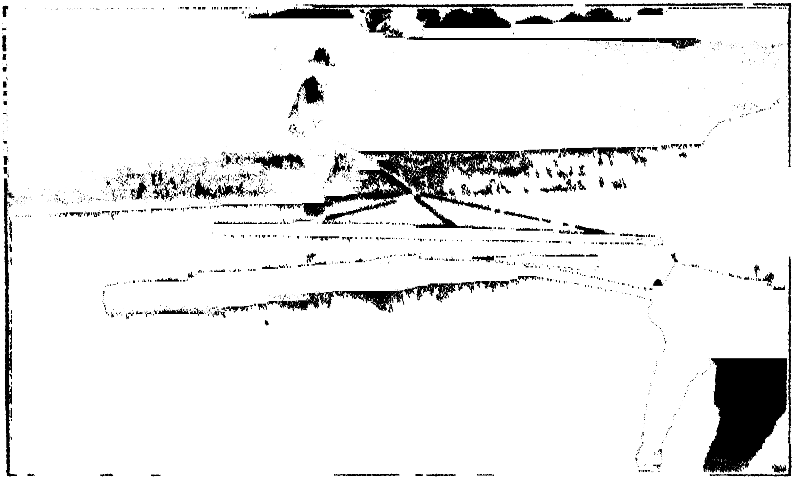


FIG 109 Operating straightedges on concrete pavement to remove irregularities and scrape off excess mortar

The finishing machine is followed by several operations performed by hand, the first being floating with a longitudinal float This float has a wooden surface, 6 to 10 in wide and 12 to 18 ft long, which is stiffened to prevent bending and warping and is fitted with handles so that it may be operated by two men standing on bridges which run on wheels on the side forms The float is operated longitudinally with a short transverse motion and each floating overlaps the preceding floating by one-half the length of the float The longitudinal floating is usually followed by another floating with straightedges mounted on long handles, which remove the thin layer of mortar on the surface and serve further to improve the longitudinal smoothness of the pavement A straightedge about 10 ft long, and mounted on a long handle, is then held over and very slightly above the pavement to check irregularities in the surface, successive straightedging overlaps one-half the length of the straightedge High spots are removed with straightedges and low areas are filled, the corrected areas are then smoothed with floats and again straightedged.

The final finishing usually is performed with a belt moved longitudinally along the surface with a short transverse motion, this is done after the water sheen on the concrete surface has disappeared, and it produces a slightly gritty surface which is uniform in appearance and increases the resistance to skidding

Courtesy of Portland Cement Association

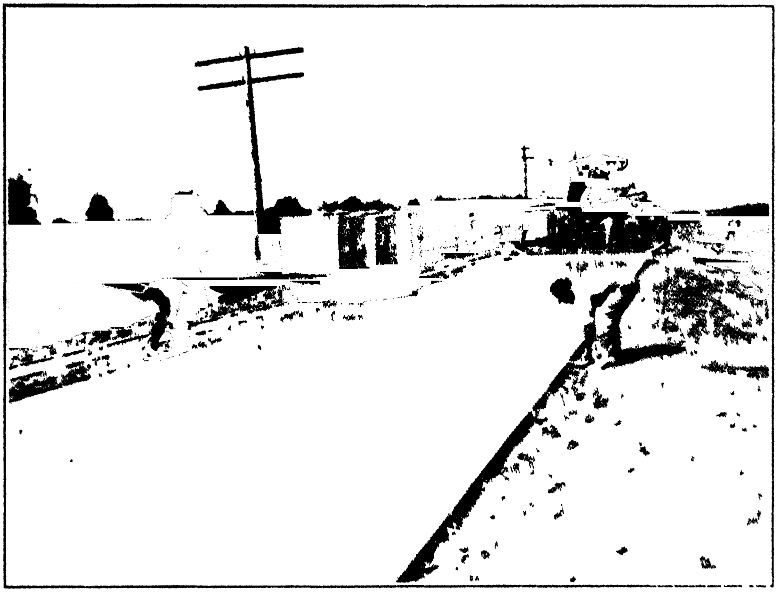
Fig 110 Finishing a concrete pavement with a longitudinal float

Some highway departments prefer a somewhat more non-skid surface than can be obtained with a belt, and broom the concrete surface transversely as the final finishing operation Bass or bassine fibered brooms, at least 18 in wide, and with fibers not over 4½ in in length, are commonly used, and the brooming is performed with one continuous stroke from the center to the edge of the pavement and in such a way that the depth of the broom marks do not exceed $\frac{1}{16}$ in

It is desirable to round the upper edges of the pavement and joints with an edging tool.

Curing the Concrete. Freshly placed concrete pavement slabs are protected against drying out for a definite period of time after initial hardening has occurred in order to obtain the desired strength and resistance to wear. This treatment, called curing, is conducted in two stages known as preliminary and final curing.

Preliminary curing is conducted by covering the concrete surface with burlap blankets as soon as the concrete has hardened sufficiently so that it will not be marked by the blankets and applying water in the form of a fine spray.



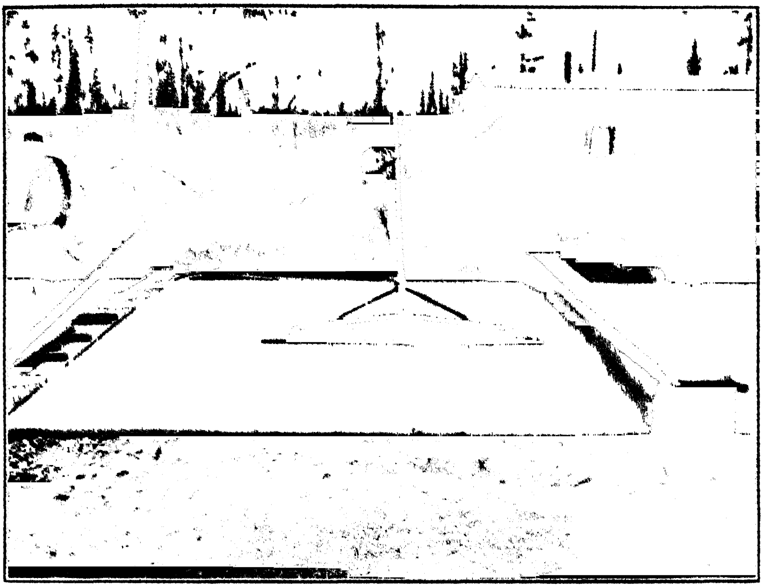
Courtesy of Portland Cement Association

FIG. 111. Final finishing of a concrete pavement with a belt.

Final curing is begun when the concrete has become hard enough to eliminate the possibility of any damage by walking on the slab, removing the forms, or handling the devices used for curing. A large number of curing methods have been, and are, practiced, including the use of sprinkling, ponding, moist earth, wet straw, wet mats, impermeable paper, waterproof membranes, and calcium chloride.

Sprinkling is one of the oldest methods of curing and is satisfactory when done thoroughly and continuously. It is best performed by means of an arrangement of piping and nozzles which effectively covers the entire area of pavement. It is wasteful of water and requires a large investment in piping and nozzles; therefore, it generally is applicable to small jobs only where a large supply of water is available.

The ponding method of curing is conducted by building both longitudinal and transverse dikes along the pavement and flooding the ponding sections thus created This method is applicable only when pavement grades are relatively flat; it is not generally used on street pavements Ponding is very effective when thoroughly performed because it removes all doubt in regard to the completeness of curing; it is undesirable, however, on some subgrades either because of the quantity of water required or because of the effect of saturation of the subgrade on possible subgrade movements



Courtesy of Portland Cement Association

FIG 112 Finishing a concrete pavement at an expansion joint by means of a split float

Moist earth and moist straw coverings provide an effective method of curing, provided that the covering is kept well moistened Both earth and straw coverings are somewhat costly; they also have the disadvantages that straw may blow away, and the earth covering may cause mud to be spattered and tracked over adjacent pavements, sidewalks, lawns, and other property.

Burlap, cotton, and other fiber mats are widely used for curing and are effective when kept wet. They may be covered with a waterproof material to reduce evaporation.

The use of waterproof paper for curing is permitted by some highway departments. The paper is applied directly to the surface of the concrete after the burlap blankets for the preliminary curing are removed. The paper is applied in sheets transversely across the pavement with overlapping edges, and must be weighted-down.

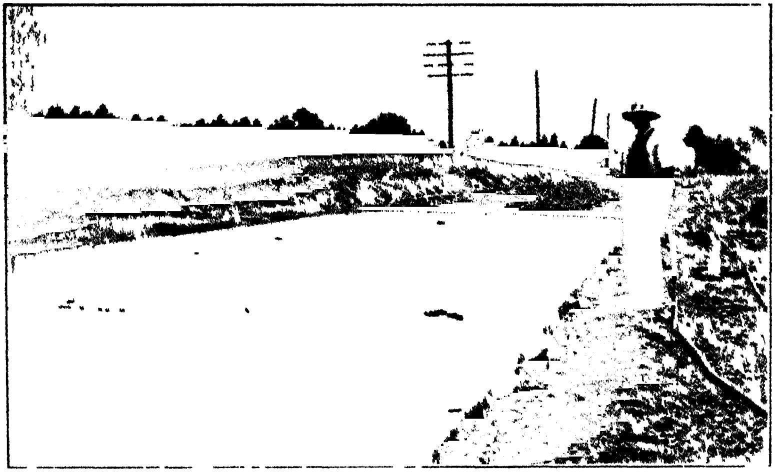


FIG. 113 Curing concrete pavement; wet burlap in foreground; wet straw covering in background

An impervious membrane is obtained by the application of some liquid which seals the surface of the concrete pavement and prevents the evaporation of water. Sodium silicate, cut-back asphalt, and emulsified asphalt have been most widely used. Sodium silicate (waterglass) possesses the advantage of being colorless.

Calcium chloride is used for curing by placing it directly on the surface or incorporating it in the concrete mixture by adding it to the mixing water. The latter method is most generally favored, and about 2 lb of calcium chloride is required per sack of cement. About 2 lb of calcium chloride per sq yd is needed when applied directly to the surface of the pavement.

The curing method to be chosen on a specific project depends upon a number of conditions. The wet-mat method is common on rural pavements. Sprinkling is quite extensively used on street pavements where practicable.

Water-curing usually is conducted for a period of 72 hours. There are numerous conditions, however, in the opinion of the author, under which water-curing should be continued for a full week, or longer. When temperatures below 45° to 50° F are encountered, water-curing is generally omitted. Impervious paper blankets or straw coverings are desirable, however, when temperatures fall below 45° F.

Removal of Surface Irregularities. Most specifications require concrete pavements to be checked for surface regularity, and the best time to do this is on the morning after placing when the burlap mats have been removed. The surface may be checked by an ordinary straightedge 10 ft or more long, but a frame mounted on wheels and carrying a contact shoe or roller midway between the wheels and connected with an electric bell circuit for registering high areas is more satisfactory. High areas are removed by grinding machines. Variations up to $\frac{1}{8}$ in. in 10 or 12 ft are generally permitted.

FIG. 114. The ponding method of curing a concrete pavement.

Removal of Forms and Protection of the Pavement. Forms generally may be safely removed on the morning after the placing of the concrete. Pavements must be protected from injury during the curing period, particularly during the first three to six days. Closing of the road or street to traffic will usually eliminate opportunities for damage. Ordinarily provision must be made on rural roads for access to adjacent lands by means of temporary driveways along the shoulders or roadsides. Intersections of other roads and streets require provision for cross-traffic. Temporary bridging is often placed to carry the traffic over the new pavement. The use of high-early-strength Portland cement may be warranted at many intersections, particularly on city streets.

Construction on Horizontal Curves. When horizontal curves are widened as well as superelevated, the entire width may be paved at one time by dispensing with the finishing machine and finishing by hand, or the normal width may be finished with the finishing machine and an additional form be placed for the widened area, which is finished by hand. When the latter method is used, the interior form must be removed and the space it occupied be filled with concrete as quickly as possible in order to prevent the formation of a weakened plane in the concrete slab. Sometimes the widened area is placed after the normal width of the pavement has been constructed; with this method, dowels may be placed at the adjoining edge of the normal section and bent inside of the form; they may be straightened into place when the widening strip is placed. Widened strips placed by this last method should not be terminated in points, but the ends should be squared off to a minimum width of about 8 to 10 in.

When finishing machines are operated on curves, the normal crown is generally retained, for two reasons: first, a flat crown frequently results in "dishing" of the center of the pavement after the longitudinal floating and final belting, whereas the normal crown is only slightly flattened; and second, changing the crown on the finishing machine screeds is inconvenient and slows up the work very materially.

Construction of Street Intersections. The construction of street intersections requires a great deal of care to provide good drainage and satisfactory riding qualities. An intersection usually includes a flat surface and a warped surface, and the correct elevations are best attained by means of carefully set stakes. The surface contour may then be secured by means of templates and long-handled floats.

Construction of Intersections on Rural Highways. Concrete pavements are carried through many rural road intersections with the normal crown and without widening. In many cases, however, an intersection is widened and flattened on account of the present or future importance of the intersecting road. Such intersections usually are constructed in advance of the normal paving in a manner similar to that at street intersections. Dowels frequently are placed at the intersecting ends to facilitate connecting the new pavement to the old when the intersecting road is paved; in other cases, the ends are thickened to provide additional strength.

Cold Weather Construction. The construction of concrete pavements is usually suspended in cold climates during cold seasons and during periods of cold weather in the spring and fall of the year. At such times the usual practice is to suspend placing when the temperature is falling and reaches 40° F.

Construction during cold weather sometimes is unavoidable, and when it is necessary or desirable, high-early-strength Portland cement should be used, the water (and the aggregates, if necessary) should be heated, and the pavement should be covered as soon as possible with burlap or cotton mats and a thick layer of straw. The opening of the pavement to traffic may be deferred as an additional precaution.

MAINTENANCE OF CONCRETE PAVEMENTS

The maintenance of concrete pavements consists of the care of joints and cracks, the replacement of pavement cuts and broken areas, and the correction of settlement.

Joint and Crack Protection. Joints and cracks should be cleaned and filled with bituminous material at regular intervals of about six to eight months. The paving grades of asphalt, ranging from 30 to 100 penetration (at 70° F), the grade depending largely on the climate, are very satisfactory for this purpose. If the openings are wider than about $\frac{1}{8}$ to $\frac{1}{4}$ in., a mixture of asphalt and sand may be used. The bituminous material should not be poured in a wide strip but should be confined to the crack as closely as possible and should be covered with sand immediately to prevent tracking.

Replacements. Replacement of portions of concrete pavements are necessary when cuts are made for the installation of utility pipes or conduits and when breaks are caused by excessive expansion, unusual loads, inadequate subgrade support, washouts, or other special conditions.

Defective subgrades should be improved by drainage, by removal and replacement of the soil, by means of sub-bases, or by other means indicated by the nature of the subgrade failure. When large areas are involved, the replacement of the slab area follows the usual construction procedure except that high-early-strength Portland cement may be used and the new slab is finished by hand. Replacement generally can be terminated at transverse joints.

When pavement cuts or comparatively small areas of damaged concrete are involved, rectangular patches should be used wherever possible. The upper 2 in. of each edge of the old concrete should be trimmed with chisels to as nearly vertical a face as possible to prevent spalling after the repair has been completed. The remaining depth should be rough, but all loose materials should be removed.

High-early-strength Portland cement is desirable for the new concrete mixture, although patching can be performed successfully with a very dry mixture made with standard Portland cement. A dry concrete mixture is used with either type of Portland cement and is tamped in place. The faces of the old concrete should be wetted and a thin mixture of cement and water applied with brushes. The concrete then is placed in layers about 2 in. in thickness and is thoroughly tamped, special care being taken to tamp the fresh concrete adjacent to the edges of the old concrete. The last layer of concrete should, after tamping, be about ½ in. above the adjacent slab. After shrinkage has taken place, the concrete again is tamped to force it into contact with the edge faces of the old concrete, and is smoothed to a true surface by means of a straightedge and floated with a wooden float.

Cuts over trenches should be wider than the trench to provide a shoulder of firm subgrade on each side of the trench for a distance of not less than 8 in , and it is good practice to make the patch about 4 in. thicker than the adjacent slab and ram the fresh concrete under the adjacent slabs for a distance of not less than 4 in. Reinforcing bars may be placed across the trench to strengthen the slab.

Correction of Settlement. Concrete pavements often settle over weak areas of subgrade which may not be detected and stabilized prior to construction. Replacement of these settled areas may be quite expensive, and raising by means of a "mud-jack" is a less costly method of repair. This device consists of an air-tight chamber in which a rather wet mixture of loam, Portland cement, and water is placed, the chamber closed, and the mixture forced (by air pressure) through a hose inserted in holes drilled through the pavement. The loam should contain from 8 to 18 per cent of clay, 10 to 40 per cent of silt, 50 to 75 per cent of sand, and 2 to 6 per cent of organic matter. The mixture consists of about 1 part of cement to 20 of earth, mixed with about 40 per cent of water; the cement reduces the shrinkage of the soil after it dries out under the concrete slab. It is necessary to drill holes spaced about 8 ft each way, and all holes must be plugged except the hole being filled. After filling with the mixture, the holes are refilled with fresh concrete or with a concrete plug which is cemented into place. This method of raising concrete slabs is effective and economical for pavement settlements up to 15 in., provided that the pavement is not badly cracked.

Bituminous patches are also used to even up settled areas; fine mixtures of premixed material or rock asphalt are satisfactory for thicknesses below 2 in. Coarse mixtures may be used in one layer for thicknesses up to 3 or 4 in and in two or more layers for greater thicknesses. Portland-cement concrete is desirable for thicknesses over 4 in; the pavement must be cut at the ends of the patch beyond the area of settlement to provide for a satisfactory connection of the old and new concrete When the settlement is greater than 10 or 12 in, a compacted earth fill may be placed between the old subgrade and the new pavement

Repairing Scaled Areas. When scaling of concrete pavements occurs, the use of bituminous patches is about the only satisfactory method of repair The concrete surface should be thoroughly cleaned of all scale by means of wire brooms and be given a light application of light cut-back asphalt or emulsified asphalt, after which a thin layer of rock asphalt or fine-graded asphaltic concrete is spread and rolled

CONCRETE FOUNDATION OR BASE COURSES

Concrete base courses are used extensively to support bituminous and block types of wearing surfaces for both streets and roads Concrete base courses differ from concrete pavements principally in regard to the quality of concrete and the installation of joints

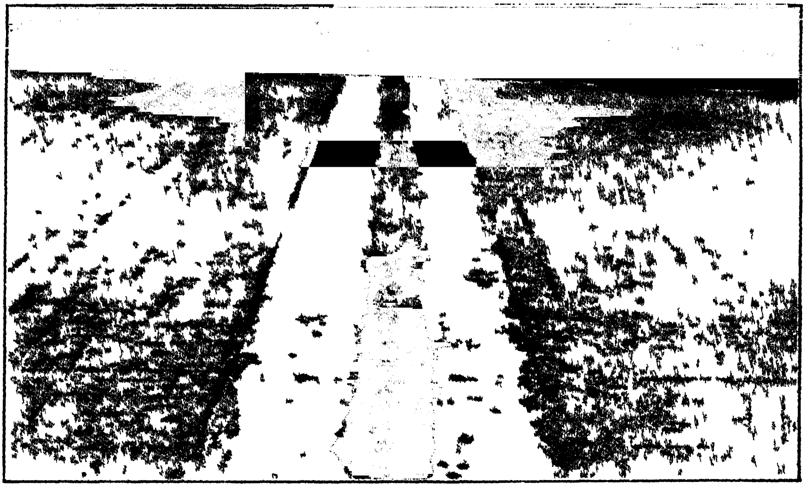
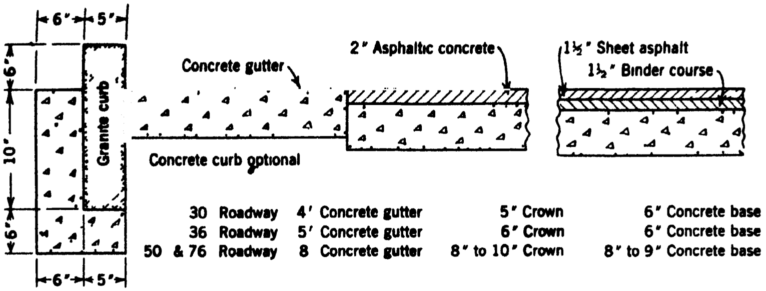
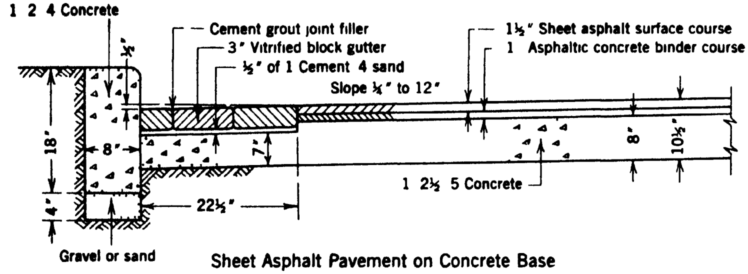


FIG 115 Concrete headers adjacent to an expansion joint in a brick pavement

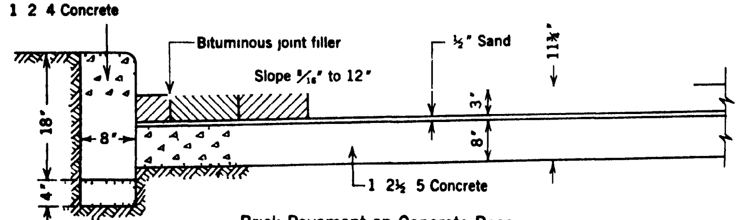
Quality of Concrete. Some difference in opinion exists in regard to the quality of concrete which should be used in concrete foundations or base courses Some engineers believe that a comparatively weak concrete is best because its lower coefficient of expansion will reduce


6"
5"
6"
10"
6"
Granite curb
Concrete gutter
2" Asphaltic concrete
1½" Sheet asphalt
1½" Binder course
Concrete curb optional
30 Roadway 4' Concrete gutter 5" Crown 6" Concrete base
36 Roadway 5' Concrete gutter 6" Crown 6" Concrete base
50 & 76 Roadway 8 Concrete gutter 8" to 10" Crown 8" to 9" Concrete base

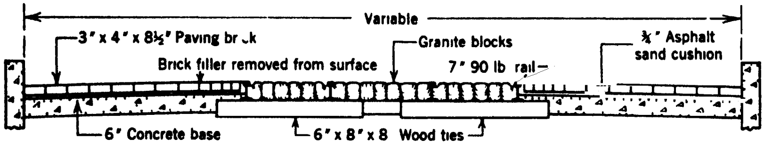
Sheet Asphalt and Asphaltic Concrete Pavements on Concrete Base
City of St Louis


1 2 4 Concrete
½"
Cement grout joint filler
3" Vitrified block gutter
½" of 1 Cement 4 sand
Slope ¼" to 12"
1½" Sheet asphalt surface course
1 Asphaltic concrete binder course
18"
8"
7"
4"
22½"
8"
10½"
1 2½ 5 Concrete
Gravel or sand

Sheet Asphalt Pavement on Concrete Base
City of Philadelphia

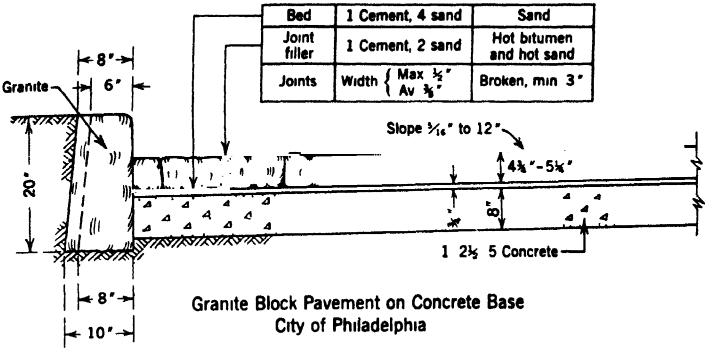


Brick Pavement on Concrete Base
City of Philadelphia



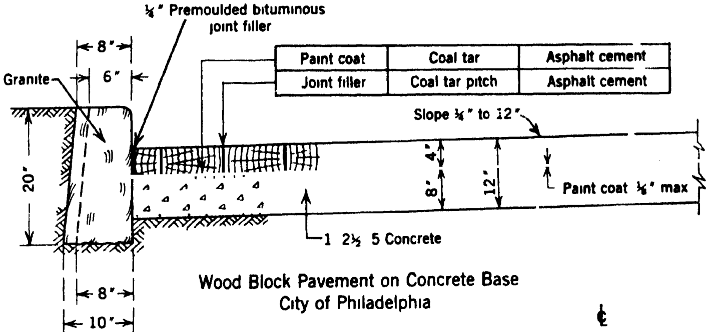
Brick Pavement on Concrete Base
City of Minneapolis

Fig 116 Typical cross-sections of pavements consisting



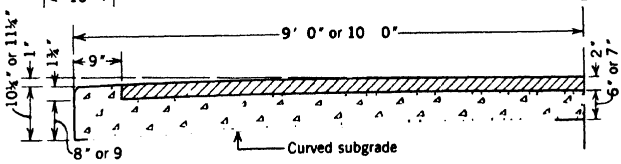
Bed	1 Cement, 4 sand	Sand
Joint filler	1 Cement, 2 sand	Hot bitumen and hot sand
Joints	Width { Max ½" Av ⅜"	Broken, min 3"

Granite Block Pavement on Concrete Base
City of Philadelphia

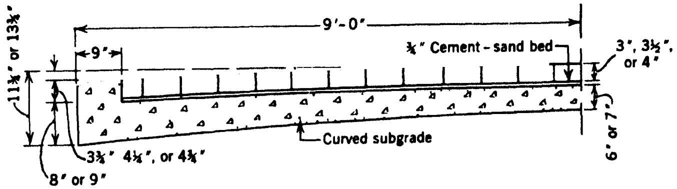


Paint coat	Coal tar	Asphalt cement
Joint filler	Coal tar pitch	Asphalt cement

Wood Block Pavement on Concrete Base
City of Philadelphia



Bituminous Pavement on Concrete Base
Pennsylvania Department of Highways



Brick Pavement on Concrete Base
Pennsylvania Department of Highways

of concrete base courses and various wearing courses the amount of warping and longitudinal expansion. The effect of a wearing course on the reduction of transverse warping of a concrete base course in comparison with a concrete pavement is illustrated in Fig. 84. On the other hand, too lean a concrete mixture on moist subgrades in cold climates is objectionable for the reason that water may enter and damage the base by freezing. Other engineers believe that the wearing course on a concrete base does not contribute very materially to the structural strength of the pavement and that the same quality of concrete therefore is needed in a concrete base course as in a concrete pavement. The fact that a concrete base is not subjected to wear, and the desirability of reducing the expansion and contraction of the concrete, are, in the opinion of the author, sufficient reasons for using somewhat weaker concrete mixtures in base courses than in pavements in locations where unsuitable subgrades are not encountered. Generally, where this practice is adopted, concrete having a compressive strength of 2500 to 3000 lb per sq in. at 28 days is used; a water-cement ratio of 6.75 to 7 50 gal of water per sack of cement is frequently specified.

Joints in Concrete Foundation Courses. The necessity of having longitudinal warping joints in concrete foundation courses at spacings of less than 20 ft is somewhat questionable on account of the protection contributed by the wearing course, and these joints are frequently omitted on two-lane pavements. The joint layout for street pavements shown in Fig. 98 could be applied to concrete foundation courses by spacing the longitudinal joints at distances of 20 ft, instead of 10 ft, from the center line. Some cities use a doweled longitudinal joint on the center line, but a free joint is desirable on widths of 30 ft or more. A free joint generally requires marginal curbs or headers to retain and protect the wearing course.

The installation of expansion joints in concrete bases is more difficult than in concrete pavements on account of the effect of the overlying wearing course. Premolded bituminous joints tend to extrude and lift up the wearing course, which then breaks up under the action of traffic; poured joints do not provide sufficient support and, if too wide, may result in the formation of two contraction cracks when a bituminous wearing course is used. These conditions may be greatly reduced by installing expansion joints at relatively short intervals (40 to 60 ft), which reduces the necessary width of the joints, and by placing wooden boards, cork, and air-chamber joint fillers slightly below (⅜ to ½ in.) the upper surface of the base. An open type of joint, covered with a 16-gauge metal plate and sealed with burlap and bituminous material, has been used in concrete bases for brick pavements in Summit County, Ohio. Figure 115 shows a joint used by the Division of Highways, Illinois Department of Public Works, in which the adjacent sections of base are completely separated and have thickened ends to support loads; a rather wide, bituminous-filled joint is place at intervals of about 200 ft.

Contraction joints are desirable in concrete base courses and should be spaced in accordance with the principles governing the spacing of contraction joints for concrete pavements. When expansion joints are closely spaced (at intervals of 40 to 60 ft), contraction joints are spaced half way between expansion joints.

Cross-Sections of Concrete Base Courses. The thickness of concrete base courses may be determined by the methods described in Chapter XI, but this design procedure has not been widely followed up to the present time. Uniform thickness of concrete base courses has been typical of cross-sections of municipal street pavements. Several typical sections of concrete base courses are shown in Fig. 116.

Construction of Concrete Base Courses. The construction of concrete base courses follows essentially the same procedure as that for concrete pavements. Heavy steel side forms are required, and machine finishing is desirable. Marginal and other integral curbs are constructed, as for concrete pavements, by having supplementary forms and placing the concrete immediately after the finishing of the base concrete.

Surface Finish. Concrete base courses for block types of wearing surfaces have a smooth surface finish. A slightly roughened surface is frequently used in connection with bituminous wearing courses, and may be obtained by brooming the surface of the concrete, after setting has begun, by means of a coarse-fiber or steel broom operated transversely across the base course.

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CHAPTER XIII

SHEET ASPHALT AND ASPHALTIC CONCRETE PAVEMENTS

Sheet asphalt is a mixture of sand, fine mineral matter (called mineral filler), and asphalt cement. Asphaltic concrete consists of a mixture of fine aggregate, coarse aggregate, and asphalt cement. The fine aggregate consists of sand or a mixture of sand and mineral filler. The coarse aggregate usually consists of crushed stone or crushed slag, but both crushed and uncrushed gravel are sometimes used The No. 10 sieve marks the dividing line between the fine and coarse aggregates. Most sheet asphalt and asphaltic concrete mixtures are mixed hot, but some mixtures may be laid cold.

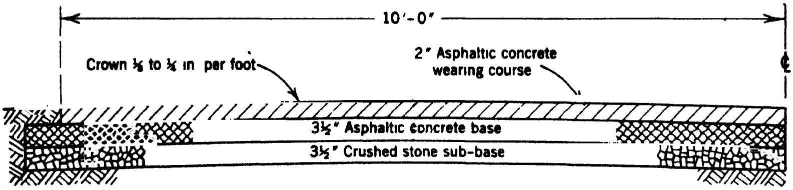
10'-0"
Crown ⅛ to ¼ in per foot
2" Asphaltic concrete wearing course
℄
3½" Asphaltic concrete base
3½" Crushed stone sub-base

FIG. 117. Typical cross-section of asphaltic concrete pavement on an asphaltic concrete base

Foundations and Cross-Sections. Sheet asphalt and asphaltic concrete pavements are placed on concrete, waterbound macadam, bituminous macadam, and asphaltic concrete foundations or base courses. Less frequently they have been constructed on gravel, traffic-bound macadam, and dry-rolled broken-stone base courses. Reconstructed pavements of many types (including block pavements) are often used as base courses for new sheet asphalt or asphaltic concrete wearing courses. Prime coats or tack coats are extensively used on base courses of waterbound macadam, gravel, and old bituminous pavements prior to the construction of the wearing course.

Binder courses are intermediate layers of asphaltic concrete which are placed on concrete base courses to provide a satisfactory bond between the base course and the bituminous wearing course. They are almost always used below a sheet asphalt wearing course and occasionally below fine-graded asphaltic concrete wearing courses. They are unnecessary on flexible base courses, or on concrete base courses when a coarse-graded asphaltic concrete wearing course is constructed. They generally are 1½ in. in thickness.

Typical cross-sections of sheet asphalt and asphaltic concrete pavements on concrete bases are shown in Fig. 116. A typical cross-section of an asphaltic concrete pavement on an asphaltic concrete or "black" base is shown in Fig. 117. The principles governing the structural design of flexible pavements described in Chapter VII may be applied to the determination of the thickness of asphaltic concrete and sheet asphalt on flexible types of foundation courses. The thicknesses so determined apply to the total thickness of the foundation course, or courses, and the wearing course.

SHEET ASPHALT MIXTURES

Mechanics of Sheet Asphalt Mixtures. Sheet asphalt must possess stability to resist deformation when used as a wearing course in a pavement, and this stability is the result of two conditions: first, the ability of the asphalt cement to bind together the particles of mineral aggregate within the range of temperatures which will normally be encountered in the pavement; and second, the amount of frictional resistance which may be developed between the particles of the mineral aggregate. Some engineers believe that, with a sharp sand, interlocking of the particles occurs to some extent to give additional stability to the sheet asphalt mixture.

The binding power of the asphalt cement is controlled by selecting a grade of material of the proper consistency. The frictional resistance between the mineral particles depends upon (1) the total volume and size of the void spaces in the mineral aggregate in the condition obtained after compaction of the pavement mixture; (2) the relation between the bitumen content of the mixture and the volume of void spaces in the mineral aggregate; (3) the thickness of the film of bitumen on the particles of mineral aggregate; (4) the shape and nature of the surface of the particles of mineral aggregate; and (5) the degree of compaction of the mixture in the pavement.

If a sand, or any mineral aggregate, were composed of spherical particles all of the same diameter, and these particles were arranged in the most compact manner possible, the volume of voids would equal 26 per cent of the total volume occupied by the particles. If the size or the diameter of these particles were decreased, the volume of voids would remain the same, but the size of each individual void space would be reduced. At the same time, the number of points of contact between particles in a given volume would be increased. If we should now place smaller spherical particles so that they just touched the larger particles, both the volume and the size of the void spaces would be further reduced and the number of points of contact would again be increased. The desirable limits of void content and void size are obtained by selecting sands composed of relatively small grains and combining with them very finely divided rock dust in suitable proportions. The limit to which this selection and combination can be carried out will depend upon the degree of stability necessary and upon the "workability" of the mixture.

It was the early belief that the amount of bitumen necessary in a sheet asphalt mixture depended upon the total surface area of the grains to be covered with a film of the bitumen. The surface area of the aggregate probably affects the amount of bitumen to some extent, but it has been conclusively shown that the void content of the aggregate and the workability of the mixture are the controlling factors. If we accept the theory that the stability of sheet asphalt mixtures depends to a large extent upon the frictional resistance of the mineral grains against each other, it follows that the bitumen content should be equal to or slightly less than the void content of the aggregate after compaction of the mixture has taken place. An excess of bitumen will decrease or eliminate frictional resistance between the aggregate particles and will place dependence for stability on the cementing, or binding, value of the bitumen. On the other hand, a great deficiency in the amount of bitumen may cause raveling, cracking, and disintegration of the pavement. It seems desirable to use as much bitumen as possible just so long as the voids are not overfilled and the bitumen does not "flush" or exude on the surface during compression, and the mixture can be spread, raked, and rolled satisfactorily, or, in other words, is *workable.*

The thickness of the film of bitumen on the particles of mineral aggregate in a sheet asphalt mixture is dependent upon the temperatures of the mineral aggregate and the asphalt cement at the time of mixing. The mixing temperatures are limited by mechanical conditions and under average circumstances vary from 225° to 375° F. No attempt is made to increase or decrease materially the film of bitumen from the condition obtained with these temperatures, as the workability of the mixture determines the mixing temperature, and the resulting film thickness will, under normal circumstances, be satisfactory.

Mineral particles that are round or spherical produce points of contact, whereas irregularly shaped particles tend to produce planes of contact, thereby increasing the amount of friction between the grains. The particles should preferably have rough or pitted surfaces in order to develop high resistance to displacement. This condition is also desirable for the reason that bitumen adheres better to rough than to smooth surfaces.

The stability of sheet asphalt pavements necessarily is dependent upon the degree of compaction secured during construction, and the greatest care in the selection and proportioning of the materials will not compensate for lack of thorough compression.

Mineral Aggregate. The mineral aggregate for sheet asphalt mixtures consists of a mixture of sand, which furnishes the coarser particles, and rock dust, generally called mineral filler, which furnishes the finer particles.

Sand. The sand should be hard, sound, and clean, and, as has previously been explained, the grains should preferably be irregular in shape and have rough surfaces. The sand used in the early sheet asphalt pavements varied greatly in mesh composition. The first attempt to establish limits for mesh composition was made by Clifford Richardson, who, as the result of extensive studies and investigations of sheet asphalt pavements, recommended the following sand gradings for light and heavy traffic.

		Per Cent			
Passing	Retained on	Light Traffic		Heavy Traffic	
10-mesh sieve	20-mesh sieve	7	30	5	23
20-mesh sieve	30-mesh sieve	10		8	
30-mesh sieve	40-mesh sieve	13		10	
40-mesh sieve	50-mesh sieve	15	48	13	43
50-mesh sieve	80-mesh sieve	33		30	
80-mesh sieve	100-mesh sieve	11	22	17	34
100-mesh sieve	200-mesh sieve	11		17	
200-mesh sieve		0		0	

Pavements constructed with sands having these mesh compositions were uniformly successful, and these sand gradings were widely adopted and generally became known as "standard" gradings. Extensive investigations and the construction of many pavements have demonstrated the fact, however, that many sand gradings may be successfully used. The presence of a considerable quantity of grains that will pass the 10-mesh sieve and be retained on the 40-mesh sieve is desirable; these grains appear to add supporting power to the pavement, and they seem to have a marked influence on the workability of the mixture. The grains between 80-mesh and 200-mesh in size have a great influence on the size of the voids, and most authorities believe that sands should contain at least 16 to 20 per cent of this size. The grains below 200-mesh in size are not of value; their presence in considerable quantities tends to produce a "mushy" mixture.

The following specification for sand is recommended by the Asphalt Institute:

Passing	Retained on	Per Cent	
	10-mesh	0 to 2	
10-mesh	20-mesh	3 to 15	14 to 50
20-mesh	30-mesh	4 to 15	
30-mesh	40-mesh	5 to 25	
40-mesh	50-mesh	5 to 30	30 to 60
50-mesh	80-mesh	5 to 40	
80-mesh	100-mesh	6 to 25	16 to 40
100-mesh	200-mesh	6 to 25	
200-mesh		0 to 5	

This specification places a limit upon the sizes of the sand grains as defined by eight sieves. It also divides the mesh composition into three main groups: 10- to 40-mesh, 40- to 80-mesh, and 80- to 200-mesh, which may be considered as coarse, intermediate, and fine grains, and places limits on the groups as well as upon the individual sizes. The percentages above 10-mesh and below 200-mesh are necessary as "tolerance" limits. It is open to question whether the limitations upon the individual sizes are necessary, and many engineers place limitations on the three groups only.

Combination of Two or More Sands. The specification limits which previously have been given seem to permit a wide range of gradings; as a matter of fact, it is seldom that a single sand having a satisfactory mesh composition can be found, and two or more sands must be mixed together. The practice of mixing more than two sands is objectionable because of the difficulty of obtaining a uniform mixture and should be discouraged whenever possible. The proportions of two or more sands may be obtained by trial calculations or by graphical methods.[1] Table 27 shows the mesh composition of two sands and of a satisfactory combination.

Mineral Filler. Mineral filler, or mineral dust, is used to fill the small voids in the sheet asphalt mixture and thereby produce a mixture which is sufficiently tough and stable to withstand the action of traffic and to prevent the intrusion of moisture. Portland cement was first largely used for this purpose and is an excellent filler in every respect, but for reasons of economy it has largely been superseded by limestone dust. Hydrated lime also is a satisfactory mineral filler but is more costly than limestone dust. The size of the particles of mineral filler is very important; all the material should pass through a No. 30 sieve, and at least 75 per cent should pass through a No. 200 sieve.

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[1] A graphical method of determining the proportions of sands is described in *Circular* 15, The Asphalt Institute.

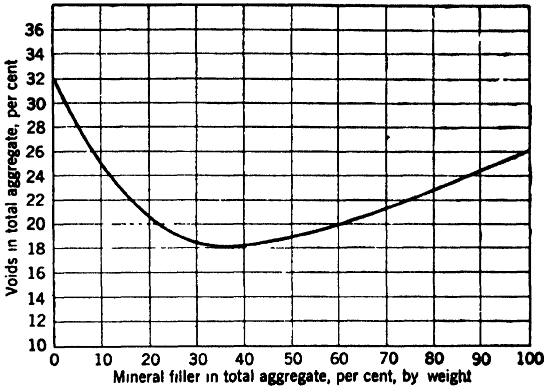


FIG. 118. Effect of percentage of mineral filler on the void content of compacted mixtures of mineral filler and sand. (*Skidmore.*)

TABLE 27

COMBINATION OF TWO SANDS TO GIVE SATISFACTORY GRADING FOR SHEET ASPHALT

	Sand A, Per Cent	Sand B, Per Cent	Combined Sand: 2 Parts Sand A, 1 Part Sand B, Per Cent	
Retained 10-mesh	2	...	1	36
Retained 20-mesh	9	1	6	
Retained 30-mesh	21	1	14	
Retained 40-mesh	21	1	15	
Retained 50-mesh	25	2	18	34
Retained 80-mesh	18	13	16	
Retained 100-mesh	3	19	8	28
Retained 200-mesh	1	58	20	
Passing 200-mesh	0	5	2	
	100	100	100	

Asphalt Cement. The use of too soft an asphalt will cause a sheet asphalt pavement to rut and shove, while a material that is too hard will become brittle in cold weather and cause cracking of the pavement. The kneading action of traffic has an important influence in keeping a sheet asphalt pavement in good condition, and the softer grades of asphalt cement are used for light and moderate traffic. The grades of asphalt recommended for various traffic and climatic conditions are shown in Table 28.

TABLE 28

PENETRATION LIMITS OF ASPHALT FOR USE IN SHEET ASPHALT PAVEMENTS

Traffic	Temperatures		
	Low	Moderate	High
Light	50–60	50–60	40–50
Moderate	50–60	50–60	40–50
Heavy	40–50	40–50	30–40

Determination of the Density and Void Content of Sheet Asphalt Mixtures. The density (or specific gravity in cgs units) of a compacted sheet asphalt mixture may be determined by weighing a sample of it in air and in water and applying the formula

$$d = \frac{W_a}{W_a - W_w} \tag{53}$$

in which W_a is the weight of the sample in air, W_w the weight of the sample in water, and d the density of the compacted mixture.

The "theoretical maximum density" of a compacted mixture of mineral aggregate and bituminous material is obtained when the quantity of bituminous material is just sufficient to fill the voids in the compacted mixture; when the specific gravity and proportion of each ingredient are known, it may be calculated by means of the formula

$$D = \frac{100}{\frac{W_1}{G_1} + \frac{W_2}{G_2} + \cdots + \frac{W_n}{G_n}} \tag{54}$$

in which D is the theoretical maximum density of the compacted mixture; and W_1, W_2, etc., the percentages by weight, and G_1, G_2, etc., the specific gravities of each ingredient. It should be borne in mind that this conception refers to a voidless mass. If a mixture, as actually compacted, contains voids, the percentage of voids may be calculated from the formula

$$V = \frac{100(D - d)}{D} \tag{55}$$

in which V is the percentage of voids, and D and d have the same significance as in formulas 53 and 54.

Proportioning Sheet Asphalt Mixtures. Clifford Richardson recommended that for heavy traffic the sand should conform to the grading previously described and that sufficient mineral filler should be incorporated to obtain 13 per cent of mineral aggregate below 200-mesh in size in the mixture of sand, mineral filler, and asphalt He found that normally such a mixture would contain 10.5 per cent of asphalt cement. For light traffic, less mineral filler was recommended. From the previous discussion of the mechanics of sheet asphalt mixtures, it is obvious that, by varying the proportion of mineral filler in the mixture, the void content of a sheet asphalt may be varied. Figure 118 illustrates the effect of various percentages of filler on the void content of a compacted mixture ot sand and mineral filler. It will be observed that the sand, itself, contained 32 per cent of voids, and that the void content of mixtures of sand and mineral filler decreased, as the proportion of mineral filler was increased, to a minimum void content of 18 per cent with 63 per cent of sand and 37 per cent of filler, and that additions of mineral filler beyond 37 per cent caused the void content to increase. Actually, a sheet asphalt mixture made with 63 per cent sand and 37 per cent of filler would tend to be unworkable at normal mixing temperatures, and the filler contents of sheet asphalt mixtures in practice are much less than 37 per cent. The amount of mineral filler (fine material below 200-mesh in size) required for heavy traffic will vary from about 12 to 20 per cent. Filler contents of more than 20 per cent have been successfully used, but they are suitable only for extreme traffic and high air temperatures. The beginner may have some difficulty in developing an accurate conception of the action of sheet asphalt mixtures under traffic, and it is important that he understand the dual nature of this action. In a paper entitled "Theory and Practice of Asphalt Paving Mixtures" before the American Society for Municipal Improvements in 1926, A. W. Dow called attention to the fact that "high stability is not the dominant factor but rather a blending of stability and pliability." Prevost Hubbard has stated the correct principles of proportioning as follows.

1. The inherent stability of a compressed mineral aggregate increases as its density approaches that of a solid, or as its percentage of voids is decreased.
2. Mineral filler is an effective stabilizing agent from the standpoint of reducing voids in the total mineral aggregate and increasing internal frictional resistance because of its relatively large surface area per unit volume.
3. Most mineral fillers in themselves contain a high percentage of voids. Their effectiveness in reducing voids in the total mineral aggregate therefore is limited and their use in excess may defeat the purpose for which they are incorporated in the mixture.
4. Percentage of voids in the compressed mineral aggregate is in no way related to percentage of voids in the total mixture as the latter is largely a function of the amount of asphalt used.
5. The amount of asphalt which may be used safely is limited by the percentage of voids in the compressed mineral aggregate. Any excess of asphalt which would tend to overfill the voids will greatly decrease stability of the mixture.
6. Assuming a satisfactory degree of stability to exist, the more asphalt present the greater will be the durability of the mixture. As the amount of asphalt is limited by percentage of voids in the compressed mineral aggregate, it therefore follows that, when the desired degree of stability can be obtained with a relatively high void aggregate, greater durability is assured than with an exceptionally low void aggregate.
7. High stability is undesirable if obtained at the expense of durability, and great durability of the mixture is of little value if the necessary degree of stability is lacking. In designing, therefore, it is often necessary to strike a balance which will insure that the mixture possesses both characteristics, to a reasonable extent.

Hubbard refers to durability of bituminous mixtures as the ability to resist cracking and disintegration caused by weathering and water absorption. In general, sheet asphalt mixtures should contain not less than 9 per cent of asphalt, and it is desirable to employ proportions which will provide for 10 per cent, or more, of asphalt.

An experienced engineer or technician generally is able to proportion sheet asphalt mixtures by trial under job conditions, but preliminary tests on small batches are useful as a guide to proportioning on the job. The trial mixtures may be tested for density and stability.

The stability of compacted bituminous paving mixtures may be defined as their ability to resist lateral flow under loads, and it depends very largely on their shearing strength. A number of stability tests have been devised, but only one, the Hubbard-Field test, has been widely employed. In this test a cylindrical specimen of the compacted bituminous mixture (either molded in a laboratory or cut from a pavement), after heating to 140° F, is placed in a cylindrical mold which is just large enough to receive the specimen and which is provided with a sharp-edged internal ring on which the specimen rests. The mold used for fine-graded bituminous mixtures is 2 in., and that for coarse-graded mixtures is 6 in. in diameter. The mold is fitted with a plunger which rests on top of the specimen. The mold containing the specimen and the plunger is placed in a testing machine, and a gradually increasing load is applied to the plunger, forcing the mixture through the orifice formed by the circular ring. The maximum load, in pounds, is taken as a measure of the stability of the mixture, and a minimum load of 2000 lb is usually considered desirable for sheet asphalt and asphaltic concrete mixtures for heavy-traffic conditions; a minimum load of 1200 lb may be used as a criterion for these mixtures for light-traffic conditions The test is conducted at 140° F because this temperature is about the maximum which prevails in pavements.

With a given sand and a given filler, small batches of sheet asphalt are prepared by hand mixing and compressed in a cylindrical forming mold under a pressure of 3000 lb per sq in. The specific gravity of each is determined by test, and the percentage of voids in each mixture is calculated. The void calculations and the appearance of the mixtures will indicate whether the asphalt content is sufficient. Mixtures with insufficient asphalt will appear dry and will show a void content greater than 3 to 5 per cent. Mixtures containing an excessive proportion of asphalt will appear "fat" or "rich" and will tend to exude asphalt when compressed. It may be desirable to start out with mixtures containing three proportions of filler to sand, say 12, 15, and 18 per cent, by weight. Three percentages of asphalt may be mixed with each of these filler-sand mixtures, say 11, 10, and 9. Some of the mixtures will be discarded on the basis of appearance alone, and it may be desirable to prepare others. The compressed mixtures are tested for stability and that mixture selected for beginning the work which has the required stability and minimum void content and demonstrates good workability.

ASPHALTIC CONCRETE MIXTURES

Asphaltic concrete mixtures may be grouped into three classes with reference to the size of the mineral aggregate, as follows:

I. Coarse-graded asphaltic concrete, in which the mineral aggregate consists of a mixture of coarse and fine aggregates with more coarse than fine. These mixtures are used for base, binder, and surface courses. Mineral filler is used in surface courses but is usually omitted in base and binder courses.
II. Fine-graded asphaltic concrete, also called stone-filled sheet asphalt, containing more fine than coarse aggregate. These mixtures are used only in surface courses.
III. One-size-stone asphaltic concrete, in which the mineral aggregate is composed entirely, or almost entirely, of coarse aggregate and asphalt cement. These mixtures are now uncommon but were once quite widely used.

Theory of Asphaltic Concrete Mixtures. The stability of an asphaltic concrete mixture depends principally upon three factors: (1) the mechanical interlocking of the coarse aggregate particles; (2) the frictional resistance between the particles of the total mineral aggregate; and (3) the ability of the asphalt to cement or bind the particles of mineral aggregate together.

The coarse aggregate of a coarse-graded or a one-size-stone asphaltic concrete mixture usually consists of crushed stone (crushed gravel is included in this designation) or a crushed slag. The particles of crushed stone or slag are angular and irregular in shape and possess the ability to lock themselves together when compressed with a roller. This condition is similar to that obtained in a plain waterbound macadam pavement. The particles of stone are locked or "keyed" together and thus give stability to the pavement. With such coarse aggregates the dependence for stability on frictional resistance between particles, and on the cementing ability of the asphalt cement, is not so great as in sheet asphalt, provided that the voids in the coarse aggregate are not overfilled so as to prevent the particles from interlocking.

Fine-graded mixtures, in which the finer particles of mineral aggregate predominate, or coarse-graded mixtures, with gravel as the coarse aggregate, depend for stability almost entirely on the frictional resistance between the aggregate particles and the cementing power of the bitumen. In this respect they are similar to sheet asphalt mixtures.

Mineral Aggregates. The Deval abrasion test is widely used as a basis for controlling the quality of crushed stone, a maximum percentage of wear of 8 and 6 being widely specified for base (and binder) and surface courses, respectively. Crushed slag is generally required to have a weight of 70 lb per cu ft. Recommended gradings of coarse aggregates for various asphaltic concrete mixtures are shown in Table 29.

The size and grading of sand for fine-graded asphaltic concrete should conform to the requirements outlined for sheet asphalt mixtures because of the dependence placed on frictional resistance of the aggregate to obtain the desired stability. The sand for coarse-graded asphaltic concrete need not be so closely restricted, and the following size and grading are recommended:

PASSING SIEVE	RETAINED ON SIEVE	PER CENT
No. 4	No. 10	0–20
No. 10	No. 40	15–50
No. 40	No. 80	22–65
No. 80	No. 200	7–40
No. 200		0– 8

Mineral filler should have the fineness described for sheet asphalt.

Asphalt Cement. The asphalt cement used in fine-graded asphaltic concrete mixtures should have the same characteristics as that used for sheet asphalt mixtures. A softer asphalt cement may be used in

TABLE 29

GRADINGS OF COARSE AGGREGATES FOR ASPHALTIC CONCRETE MIXTURES

Sieve	Per Cent Passing Sieve					Seal Coat
	Coarse-Graded Asphaltic Concrete			Fine-Graded Asphaltic Concrete Surface Course	One-Size Stone Asphaltic Concrete Surface Course	
	Base Course		Binder and Surface Courses			
2½-in.	100					
2 -in.	95–100					
1½-in.		100	100		100	
1 -in.	35– 70	90–100	90–100		90–100	
¾-in.				100		
½-in.	10– 30	25– 60	35– 60	90–100	35– 60	100
No. 4	0– 5	0– 10	0– 10	0– 15	0– 10	0– 15
No. 8				0– 5		0– 5

coarse-graded asphaltic concrete mixtures; the following grades are recommended for surface courses:

	Light to Moderate Traffic	Heavy Traffic
Northern United States	60–70	50–60
Southern United States	50–60	40–50

A softer asphalt cement may be used in binder and base courses than in surface courses, but it is somewhat inconvenient except on large projects and the same material is generally used for both the base course, or binder course, and the surface course.

A rather soft asphalt cement is used for one-size-stone asphaltic concrete. Penetration grades of 85 to 100 and 60 to 70 are satisfactory for the northern and southern United States, respectively.

Composition of Asphaltic Concrete Mixtures. The usual composition of asphaltic concrete mixtures is described in Table 30; the proportions shown may have to be varied to obtain satisfactory workability or density.

TABLE 30

Composition of Asphaltic Concrete Mixtures

Type of Mixture	Per Cent by Weight			
	Coarse Aggregate	Fine Aggregate	Mineral Filler	Asphalt Cement
Coarse-graded, base course	55–75	20–35	None	4–7
Coarse-graded, binder course	60–70	25–35	None	4–6
Coarse-graded, surface course	45–60	25–45	4– 6	5–8
Fine-graded, surface course	20–35	45–60	6–12	7–9.5
One-size stone	93–95	None	None	5–7

COMPARISON OF TYPES OF WEARING COURSES

Sheet asphalt and fine-graded asphaltic concrete wearing courses are composed of fine-grained mixtures. They have the advantage that they may be constructed to a smoother and more uniform surface contour than coarse-graded asphaltic concrete wearing courses and the disadvantage of being more slippery in wet and icy weather than coarse-graded asphaltic concrete surfaces. This slipperiness may be decreased to a large extent by rolling small-size (¾- to ¼-in.) stone (which usually is coated with asphalt cement) into the surface of the pavement while it is still hot and before it is fully compacted. The necessity of using a binder course makes sheet asphalt pavements more costly than other types, and the asphaltic concrete wearing courses, therefore, often take the place of sheet asphalt.

PREPARATION OF ASPHALTIC PAVING MIXTURES

Unloading and Storage of Materials. Mineral aggregates which are shipped by rail are usually unloaded by means of bucket cranes and stored in piles.

Asphalt is shipped in tank cars and is usually heated and pumped direct into storage tanks. Sometimes, however, it is pumped directly from cars into the heating kettles. This practice is open to the objection that railroad shipments are subject to delays and that excessive demurrage may result from delays due to weather and other conditions.

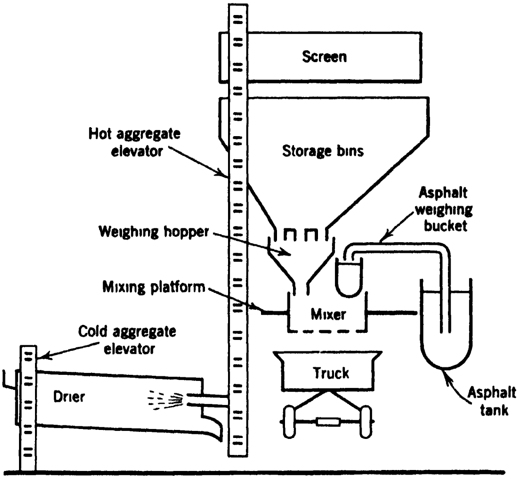


Fig. 119. Diagram of asphalt paving plant.

Paving Plants. There are three types of plants for preparing asphaltic paving mixtures: the stationary plant, which is suitable only in large cities where it can be kept in operation for long periods of time; the railroad plant, which may be moved from one city to another; and the portable plant, which was designed primarily for rural highway construction but is used to a large extent in small cities.

The various parts of a paving plant are shown in Fig. 119, including:

1. The cold-aggregate elevator.
2. The drier.
3. The hot-aggregate elevator.
4. A screening plant.
5. Storage bins for the hot aggregates.
6. Measuring hopper for weighing the aggregates.
7. Asphalt-heating kettles.
8. Asphalt-weighing bucket.
9. The mixer.

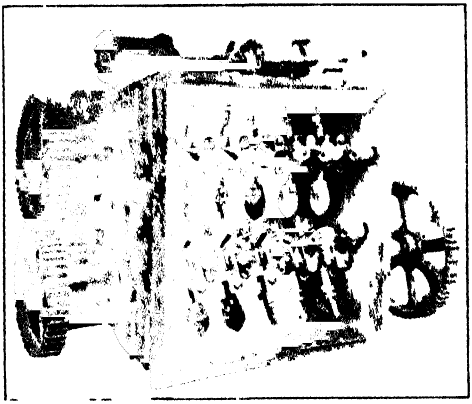
Some paving contractors prefer to eliminate the asphalt-heating kettles and transfer the asphalt directly from storage tanks to the asphalt bucket. A superheater is necessary in connection with the storage tanks in order to bring the asphalt to the proper mixing temperature.

The cold aggregates are fed in proper proportions to the cold-aggregate elevator which transfers them to the drier, consisting of a slightly inclined rotary steel cylinder so arranged that the hot gases from an oil burner dry and heat the aggregates as they pass through the drier. The dried and heated aggregates are discharged onto the hot-aggregate elevator which conveys them to an overhead screen. The separated aggregates are stored in the hot storage bins from which they are discharged into the measuring bin, located directly over the mixer. The asphalt is measured in a separate bucket so suspended that it may be readily discharged into the mixer in a thin sheet. The mineral filler generally is not heated by passing it through a drier because a large proportion of it would be removed by the draft. The mineral filler is weighed separately and generally discharged into the mixer immediately after the hot aggregate or aggregates and is heated in contact with the latter during mixing; after a uniform mixture has been attained the asphalt cement is added to the batch.

Pug Mixers. The pug mixer is most widely used for mixing asphaltic concrete and sheet asphalt mixtures; it consists of a steel box equipped with two horizontal shafts on which blades are mounted. The blades are so set that they throw the material toward the center of the mixer. The mixed material is discharged through an opening at the bottom of the mixer. The capacity of pug mixers varies from 1000 to 3000 lb per batch.

Rotary Mixers. A rotary mixer, in which the asphalt is incorporated with the aggregates under pressure, is also used. The mixer consists of a cylindrical steel drum provided with a tight-fitting cover and having shelves on the inside, similar to a concrete mixer. The asphalt is introduced into the mixer through a hollow steel shaft and is sprayed over the aggregate. The mixture is kept under air pressure usually at about 50 to 60 lb per sq in. Owing to the time required to fill and discharge the mixer, a large unit is necessary in order to obtain a large enough output to be economical. Railroad plants ordinarily have a capacity of 6000 lb, and stationary plants of much larger capacity have been built.

Temperature Control. The control of temperature is a very important feature of paving-plant operation. Asphalt kettles and storage tanks should be equipped with stationary thermometers. A registering pyrometer is usually attached to the lower end of the hot aggregate elevator so that the hot aggregates fall over the pyrometer bulb as they are discharged from the drier. The drum fireman may then regulate the heat and the feed of the cold aggregates so as to obtain proper working temperatures, which will vary, with average materials, from 250° to 375° F.



Courtesy of F D Cummer & Son Co

FIG 120 Interior of pug mixer showing arrangement of the mixer blades

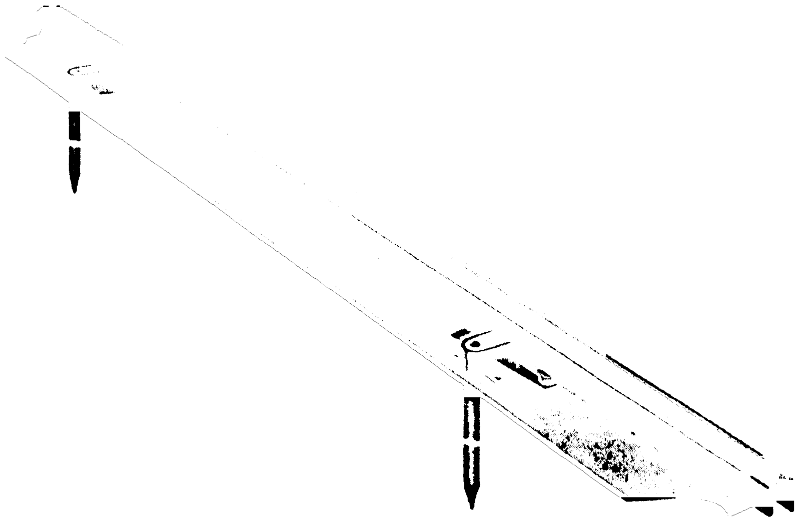
Effect of Overheating. The careful control of temperature limits, particularly of the higher limit, is necessary if injury to the asphalt is to be avoided. A very high temperature may cause burning or charring of the asphalt, rendering it useless as a cementing medium. Considerable damage, however, is possible at temperatures much below those at which actual burning or coking takes place. The operation of mixing hot mineral aggregate with hot asphalt causes changes in the physical properties of the asphalt. These changes are as follows:

1. The penetration of the asphalt is decreased, showing that hardening takes place.
2. The softening point of the asphalt is increased.
3. The ductility is decreased.

The extent of these changes is affected by the mixing temperature, and damage which is not readily apparent and which may shorten the life of a pavement thus may be caused if temperatures are not controlled carefully.

CONSTRUCTION OF BASE AND WEARING COURSES

Hauling Equipment. Trucks equipped with dump bodies are used for hauling hot asphaltic mixtures; when the mixtures will be subject to cooling, on account of long hauls or cool weather, the truck bodies are covered with canvas covers.



Courtesy of Blaw-Knox Co.

FIG. 121. Steel side form used for constructing bituminous pavements on flexible foundations.

Base courses or sub-bases should be clean and free from surface moisture when the hot asphaltic mixtures are placed.

Placing Mixtures. Asphaltic paving mixtures formerly were shoveled into place and spread by hand. Mechanical spreading machines, similar in design to the finishing machines for concrete pavements, are now used extensively for spreading asphaltic mixtures. These machines travel on side forms or on their own wheels or rollers. Some spreading machines are equipped with hoppers into which the mixtures are deposited from the trucks; others are operated in connection with separate box-type spreaders.

When mixtures are spread by hand, they are usually deposited on the foundation outside of the area to be covered with a single load, or on dumping platforms, and are shoveled into place with hot shovels and spread with hot rakes.

Courtesy of Blaw-Knox Co.

FIG. 122. Spreading asphaltic concrete with a mechanical spreader.

Base mixtures generally should be spread to thicknesses of not more than 4 in., loose, and if greater total thicknesses are to be placed, the material should be spread in two layers of approximately equal thickness.

Compacting Mixtures. Asphalt paving mixtures are compacted by means of heavy rollers, the usual weight being 10 tons. Both tandem and three-wheel rollers are used, the latter generally being preferred for initial rolling and the former for final rolling. Rolling always begins at the sides of the asphaltic course, and the initial rolling is in a longitudinal direction, the roller gradually progressing to the center of the course. The final rolling is performed in two diagonal directions, the second rolling crossing the lines of the first. Wide pavements are frequently given a final rolling at right angles to the center line.

The speed of rolling has an important influence on the density and smoothness of the base or wearing course. Generally, the speed of rollers should never exceed 3 mi per hr, and the rate of continuous rolling should be restricted to 300 sq yd per hr for binder courses, 200 sq yd per hr for base courses, and 150 sq yd per hr for surface courses.

Surface courses are checked for smoothness by means of straightedges, and defective areas are corrected by raking, removing or adding material, and rerolling.

Joints between cold and freshly laid sections of pavement are usually formed by cutting back the end of the previous work so as to expose a granular surface to which the new material may be bonded.

In order to prevent the entrance of water, it is desirable to paint the upper edges of all contact surfaces, such as curbs, gutters, and manholes, with hot asphalt cement or asphalt cement dissolved in naphtha.

Seal Coats. Seal coats are desirable on asphaltic concrete pavements of the coarse-graded aggregate type. An application of asphalt cement of ¼ to ½ gal per sq yd is made, and this is covered with mineral aggregate. The asphalt cement is applied with a pressure distributor, a hand-spray, or a squeegee buggy.

Sheet asphalt and fine-graded asphaltic concrete surfaces sometimes are given a coating of small stone which has been precoated with asphalt cement (2 to 3 per cent, by weight of the stone); the stone is distributed on the pavement, while it is still hot and before final compaction, at the rate of 8 to 12 lb per sq yd, and is immediately rolled into the surface.

Pavements of sheet asphalt and fine-graded asphaltic concrete are given a light coating of mineral filler which is brushed out thin with a fine-haired push broom. This application, which is made prior to final rolling, seals the small surface voids against moisture and, in addition, tends to cause the pavement to assume a uniform color.

Opening to Traffic. Heavy traffic, such as heavy solid-tired vehicles, should be kept off freshly laid asphaltic mixtures until they have cooled appreciably. Light pneumatic-tired traffic will not usually injure a well-compacted pavement.

Inspection. Sheet asphalt and asphaltic concrete pavements involve both manufacturing and construction operations, and careful inspection of both operations is essential. Satisfactory control of mixtures is best obtained by installing a small laboratory at the paving plant, in charge of a plant inspector. The principal duties of the plant inspector are as follows:

1. Sieve tests on each shipment of each mineral aggregate and mineral filler. When mineral aggregates are delivered by truck, one sample should be tested for each 30 to 40 cu yd of each material received.
2. Daily inspection of the weighing equipment for mineral aggregates to ascertain whether it is in proper working condition.
3. Frequent checking of the tare weight of the asphalt bucket.
4. Frequent checking of the mixing time to insure uniform mixtures.
5. Record of temperatures, at frequent intervals, of heated mineral aggregates and asphalt, and of the paving mixtures.
6. Several daily sieve tests of mixed sands when two or more sands are used.
7. Inspection of unloading and stockpiling of coarse aggregates to prevent segregation.
8. Frequent inspection of the proportioning of the mineral aggregates as they are fed to the drier, and of the measurement of mineral filler.
9. Sampling of shipments of asphalt and forwarding of samples to a central testing laboratory.
10. Record of number of loads of each mixture sent out from the plant during each day.

At the beginning of a paving project, the inspector should check the accuracy of all weighing scales. The large scales for weighing mineral aggregates may be checked by weighing two or three batches on the scales, dropping the material through the mixer into a clean truck body, and weighing it on an accurate platform scale equipped with a suitable container.

Registering pyrometers frequently show errors; they may be checked by removing the bulb from the boot of the hot elevator and immersing it in a pailful of hot sand along with an accurate mercury thermometer. The range of heating temperatures can be checked as the sand cools from the upper end of the range.

Penetration tests of incoming shipments of asphalt, and of the residues in heating kettles at the end of each day's run, are frequently made in plant laboratories. When native asphalts are fluxed at the paving plant, penetration tests of the fluxed asphalt should always be made at the paving plant unless the central laboratory is located within a few miles of the paving plant.

Careful inspection of the construction of the pavement also is essential and is in charge of a "street" inspector who should check the temperature of arriving mixtures at frequent intervals and inspect all the placing and compacting operations. The street inspector should see that the required thickness of pavement courses is obtained.

Tests of Paving Mixtures. Tests of paving mixtures include analyses to determine their composition, density tests, stability tests, and tests of the asphalt extracted and recovered from the mixtures.

Composition. The accuracy of the proportioning of the ingredients of sheet asphalt or asphaltic concrete may be determined by extracting the bitumen from a weighed sample of the mixture and making a sieve analysis of the mineral aggregate. The bitumen is extracted by means of carbon disulphide or carbon tetrachloride in a centrifuge or centrifuge extractor; when the test is conducted at plant laboratories it is desirable to use carbon tetrachloride as the solvent instead of carbon disulphide, which is explosive. Analyses of mixtures are usually conducted on "composite" samples made up from small samples taken from a number of batches during each day's run.

Density and Voids. The density test previously described in this chapter is valuable as a check on the effectiveness of rolling and in the control of the proportions and measurement of materials for sheet asphalt and fine-graded asphaltic concrete mixtures. It is not well adapted to most coarse-graded asphaltic mixtures on account of the "open" texture of these mixtures. The voids in compacted sheet asphalt and fine-graded asphaltic concrete mixtures should seldom exceed 4 or 5 per cent.

Stability Tests. Stability tests, employing the Hubbard-Field method previously described, often are made on cylindrical cores of the compacted bituminous mixtures which are carefully cut from the pavement by means of a properly shaped cutting tool.

Tests of the Asphalt Extracted and Recovered from Paving Mixtures. The hardening effect of the processing operations on the asphalt cement in sheet asphalt and asphaltic concrete mixtures may be determined by making tests for penetration, softening point, and ductility on the asphalt cement after extraction and recovery from the paving mixtures. The asphalt cement is extracted by digesting the paving mixture in benzol, which is then removed from the asphalt by distillation in the presence of carbon dioxide.

SAND-ASPHALT PAVEMENTS

Sand-asphalt pavements are similar, but generally inferior, to sheet asphalt pavements, and were developed for sandy areas where suitable local surfacing materials were not available. They have been used most extensively in North Carolina but also have been constructed in Delaware, Massachusetts, Florida, Alabama, Louisiana, and other states. They usually are constructed in two courses on firmly compacted subgrades, the base course being 3 to 4 in., and the top course 2 in., in thickness. They consist of mixtures of sand and asphalt cement for the base course, and of sand, mineral filler, and asphalt cement for the wearing course, and are prepared in standard paving plants; the sand and asphalt are heated to temperatures of 250° to 350° F before mixing in a pug mixer.

Materials. Since local sands are employed in the sand-asphalt mixtures, the range of gradings is rather wide. Typical limitations on sand grading are shown in Table 31. The bitumen content varies from 6 to 9.5 per cent in the base-course mixture and from 9 to 12.5 per cent in the wearing-course mixture. Sufficient mineral filler is used in the wearing-course mixture to provide about 7 to 12 per cent of mineral material passing a No. 200 sieve.

Construction. Side forms are used, generally consisting of 2-by-6-in. or 3-by-6-in. dressed lumber which is securely staked at 2- to 2½-ft intervals. These forms are generally left in place, especially when the soil on the shoulders cannot be firmly compacted; they finally decay but furnish side support until the pavement consolidates and hardens under the action of traffic and weather.

TABLE 31

TYPICAL LIMITATIONS ON GRADING OF SAND FOR SAND-ASPHALT PAVEMENTS

Passing	Retained on	Per Cent by Weight	
		Base Course	Surface Course
No. 10 sieve	....	95 to 100	98 to 100
No. 10 sieve	No. 40 sieve	14 to 50	14 to 50
No. 40 sieve	No. 80 sieve	30 to 60	30 to 60
No. 80 sieve	No. 200 sieve	16 to 40	16 to 40
No. 200 sieve	....	0 to 6	0 to 5

The mixtures are spread by hand methods or by means of spreading machines and are compacted by means of tandem rollers, an 8- to 10-ton roller being generally used. For the top course, 6- to 8-ton rollers are sometimes used.

A squeegee coat of 0.06 to 0.08 gal of hot asphalt cement per sq yd is applied to the surface of the base course to secure adhesion of the wearing course to the base course.

WARRENITE-BITULITHIC PAVEMENT

The Warrenite-Bitulithic pavement is a patented pavement which consists of a fine surface mixture, similar to sheet asphalt, placed directly on a coarse-graded asphaltic concrete mixture, after placing and before any rolling. The fine surface mixture thus is welded to the coarse mixture below, a much closer union being obtained than between sheet asphalt and a binder course.

COLD-LAID BITUMINOUS MIXTURES

Sheet asphalt and bituminous concrete mixtures require extensive heating and must be placed while hot. Numerous mixtures which may be placed and compacted cold have been developed; they possess many advantages for the construction of small projects and for the maintenance and repair of bituminous pavements. Most of these mixtures may be stored in piles for periods varying from a few weeks to several months without deterioration and can conveniently be shipped by rail from commercial mixing plants.

Amiesite Mixtures. Amiesite mixtures are prepared by mixing crushed stone, which has been dried, with a liquefier (consisting of naphtha, gasoline, or kerosene) and hot asphalt cement, hydrated lime being added to assist in mixing and to harden the asphalt. A binder course and a surface course are usually constructed, the former being about 1½ in and the latter about ¾ in. in thickness.

Colprovia Mixtures. Colprovia mixtures consist of mineral aggregate, flux oil, and powdered asphalt, and include coarse-graded and fine-graded asphaltic concrete, and sheet asphalt. After the mixtures are placed in a pavement, the flux oil and powdered asphalt are fluxed by rolling, by the kneading of traffic, and by the heat of the sun's rays, to form asphalt cement. Surfaces of Colprovia mixtures tend to mark under traffic when first placed but gradually harden and become smoother as they grow older.

Cut-Back Asphalt and Cut-Back Tar Mixtures. Bituminous mixtures may be made by mixing mineral aggregates and cut-back asphalts or cut-back tars as described in Chapter X. Mixtures of the sheet asphalt type are very useful for making repairs to bituminous pavements and for small construction projects. They are frequently made in concrete mixers or "cold-patch" mixers but are also prepared in commercial plants.

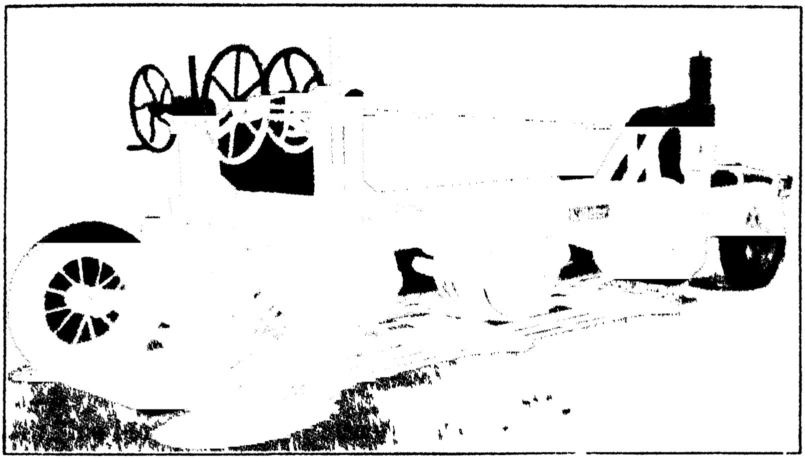
Asphalt-Emulsion Mixtures. Emulsified asphalt is substituted for cut-back asphalt or cut-back tar for cold bituminous mixtures;

it has the advantage that the mineral aggregate does not require drying.

Macasphalt Mixtures. Macasphalt mixtures are similar to Colprovia mixtures except that a hard asphalt takes the place of powdered asphalt, water is added in the mixture to retard the hardening of the asphalt.

MAINTENANCE

The maintenance of sheet asphalt, asphaltic concrete, and similar bituminous pavements consists of replacing defective areas which develop in the foundation and surface courses, leveling depressions, surface treatment, and resurfacing.



Courtesy of Killefer Mfg Corporation

Fig 123 Killefer road disc or planer

Replacement of defective areas of foundations and surface courses should correct the cause of failure and should duplicate the original construction as closely as possible. Patching of holes and other areas which require a joint between the existing pavement requires special care. The edges of the existing pavement which remains in place should be cut vertically, and the exposed cross-section should be lightly painted with rapid-curing liquid asphalt, quick-breaking emulsified asphalt, or hot asphalt cement to provide adhesion with the new material and seal the joint against the entrance of water. If cold-laid bituminous mixtures are used for the surface renewal, the new surface should be slightly higher than the existing surface after tamping or rolling to allow for further consolidation by traffic.

It is not always practicable to use hot mixtures of asphaltic concrete or sheet asphalt for making repairs to pavements of these types, and the various types of cold mixtures which have been described are especially adaptable to this work. Depressions which do not involve base failures usually are corrected by patching or placing a leveling course over the affected area. The area to be covered is given a thin paint coat of cut-back asphalt or emulsified asphalt which should be allowed to dry until sticky before placing the new material. The edges of patches or leveling courses generally are "feathered out" with a fine-graded bituminous mixture; this method is satisfactory if carefully performed under skilled supervision or by experienced workmen. The foregoing procedure may be modified when the existing surface consists of a fine-graded bituminous mixture, such as sheet asphalt, and the average thickness of the patch is greater than 2 or 2½ in. and less than 5 or 6 in. Coarse-graded mixtures, containing crushed stone as coarse aggregate in proportions of more than 50 per cent of the mixture, are inherently more stable than fine-graded mixtures, and some engineers and highway superintendents prefer to remove the existing fine-graded mixture before placing the patch or leveling course.

FIG. 121. Removing the upper portion of a rough asphaltic concrete surface by burning, prior to placing a new surface layer.

Bituminous surface treatments are well-adapted to old asphaltic concrete pavements which require attention because of wear and because of drying out of the upper surface layer. Repair of the existing surface should always precede the surface treatment, and a thin tack coat of about 0.10 gal of light cut-back asphalt (Grade RC-1) or emulsified asphalt is almost always necessary to insure obtaining satisfactory bond between the old and the new material.

Old asphaltic concrete and sheet asphalt pavements are frequently resurfaced after planing or burning the old surface. Two types of planers are used. One consists of hardened steel discs mounted on a steel frame at an angle of less than 90° with the longitudinal axis of the machine, the frame and discs being carried on two axles at a considerable distance apart. The discs shave off the pavement surface by a shearing action, and the depth of cut may be regulated. The long wheelbase is essential in order to obtain longitudinal smoothness. A machine of this type is shown in Fig. 123. Another machine consists of steel teeth set in a V-shaped frame and mounted on a long, heavy, four-wheeled chassis. A width of about 2 ft is shaved or planed as the machine is hauled along the pavement. Surface heaters (see Fig. 124) consist of inverted steel pans equipped with oil burners, mounted on trucks or trailers which are lowered to the pavement to confine and direct the heat against the surface of the pavement. The upper 1 to 1½ in. of the old wearing course is heated to a temperature which permits it to be removed with rakes or scrapers without seriously injuring the material underneath.

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CHAPTER XIV

ROCK ASPHALT PAVEMENTS

Rock asphalt is a fine-graded mixture of mineral aggregate and bitumen which may be applied either cold or heated. Wearing courses vary in thickness from about ¼ in. to 2 in. and are placed on new base courses of gravel, limerock, traffic-bound macadam, waterbound macadam, bituminous macadam, asphaltic concrete, and concrete; they are used extensively on old wearing courses and pavements which are reconstructed to serve as foundations. The thinner layers are used on flexible bases and binder courses; when placed on concrete foundations without an intermediate binder course, a thickness of not less than 1 to 1½ in. is desirable When used on low-cost foundations in areas near sources of supply, they are comparable, in cost, to various low-cost bituminous surfaces.

MATERIALS

Kentucky Rock Asphalt. Very large deposits of sandstone rock asphalt are found in western central Kentucky. The bitumen content of the material varies, and a uniform bitumen content is obtained by careful selection and mixing of material from various parts of a deposit Specifications usually provide that the rock asphalt contain from 6 to 8 per cent of bitumen, by weight. Material containing less than 6 per cent of bitumen sometimes is mixed with asphalt and is known as *processed sandstone rock asphalt.* Both hot- and cold-processed mixtures are made; the former generally are prepared in mixing plants on individual projects and are placed in the wearing course while hot; the latter are placed at air temperature and may be prepared at central plants and shipped by rail or truck.

Texas Rock Asphalt. Limestone rock asphalt is found in large quantities in Uvalde County, Texas. It consists of a shell limestone impregnated with bitumen. The natural material is crushed, pulverized, and screened through a ⅝-in. screen, and material from various portions of the deposit are mixed so that the material contains from 9 to 12 per cent of bitumen. The material is generally placed as a cold mixture, and mixtures are prepared in central mixing plants and shipped ready for use. Since the bitumen in Uvalde rock asphalt is very hard (see Chapter VI, Table 3) it is softened by incorporating a flux oil in the mixture. Two types of mixtures are common. One mixture contains 2½ to 3½ per cent of flux oil and may contain from 3 to 5 per cent of water to retard setting; the other contains the same proportion of flux oil, 20 to 35 per cent of crushed stone, and 1 to 4 per cent of water. A flux oil having a specific gravity of 0.92 to 0.95, at 77° F, and a loss on heating of not more than 5 per cent at 325° F, is used; at least 95 per cent of the crushed stone will pass a ⅝-in. screen and not more than 15 per cent will pass a No. 10 sieve. The flux oil is heated to a temperature not higher than 180° F, and the rock asphalt and crushed stone are at air temperature when mixed with the oil. The plain mixture generally is used within three weeks after mixing, but the stone-filled mixture is frequently stored for longer periods of time.

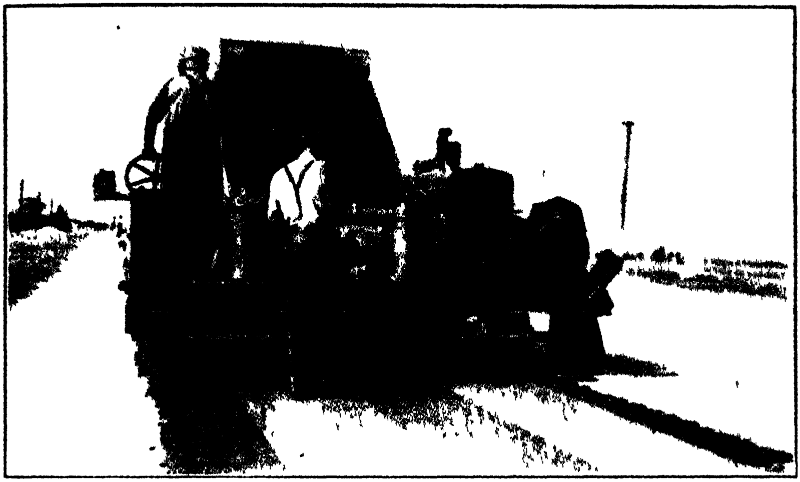
Alabama Rock Asphalt. Both sandstone and limestone rock asphalt are found in Alabama, but only the latter is produced commercially. The limestone rock asphalt is a porous limestone impregnated with 2 to 8 per cent bitumen, averaging about 4 per cent bitumen. The rock asphalt is crushed to a size having 100 per cent passing a ½-in. sieve and from 12 to 20 per cent passing a No. 200 sieve. When used as a cold-laid mixture, cut-back asphalt is added in a sufficient proportion to raise the total content of bitumen to 7.5 per cent. When used as a hot mixture, coarse sand and 85 to 100 penetration asphalt cement are added in proportions of 15 to 30 per cent and 3 to 5 per cent, respectively, by weight, of the total mixture.

Oklahoma Rock Asphalt. Sandstone and limestone rock asphalt are mined in Murray County, Oklahoma, the former containing an average of about 10 per cent, and the latter containing about 3 per cent, of bitumen. The two materials are mixed together (generally in the proportions of 70 per cent sandstone rock asphalt and 30 per cent limestone rock asphalt, by weight) and are crushed and screened to a maximum size of approximately ½ in. The mixture, which requires no fluxing material, is shipped by rail or truck to the work for which it is intended.

CONSTRUCTION

Reconstructed Foundations. When old road and street wearing courses and pavements serve as foundations for rock asphalt wearing surfaces, preparation of the base may involve widening, replacement of defective areas of the existing foundation, and the correction of surface irregularities by means of leveling courses. The last may consist of asphaltic concrete, bituminous macadam, or thin surface layers composed of an application of bituminous material covered with crushed stone of suitable size. Areas to be covered with leveling courses are cleaned by scraping and sweeping and a thin tack coat or paint coat of rapid-curing liquid asphalt (Grade RC-0 or RC-1 is satisfactory) or quick-breaking emulsified asphalt.

Tack Coats and Binder Courses. Binder courses, composed of asphaltic concrete, are frequently used on concrete foundations and on old block or other types of pavements used as foundations. Tack coats of rapid-curing liquid asphalt or quick-breaking emulsified asphalt are applied to foundations before placing binder courses, or, if a binder course is omitted, before spreading the rock asphalt layer; the quantity of bituminous material is small as the purpose of the tack coat is merely to cause the surface course to adhere to the base. A tack coat usually consists of 0.05 to 0.15 gal of bituminous material per sq yd, and uniformity of the rate of application is very important. The rock asphalt is applied after the tack coat has become sticky but not hard and dry.



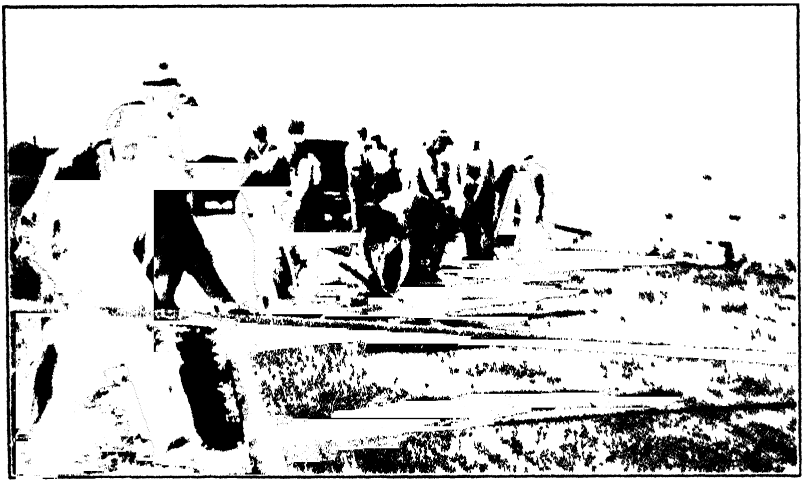
Courtesy of Kentucky Rock Asphalt Institute.

FIG 125. Spreading rock asphalt by machine

Spreading Rock Asphalt. Rock asphalt wearing courses are spread by machine spreaders, by multiple-blade drags, by motor-propelled blade graders, and by hand methods.

Machine spreaders are now used for spreading rock asphalt when the quantity of material exceeds 40 lb per sq. yd. Both types of machines described in Chapter XIII are used. Some types of machines are equipped with fluffing rakes for breaking up lumps and raking the loose material to a uniformly loose condition. The material is deposited from trucks into spreader boxes or hoppers which spread it on the foundation where it is struck off by the machine spreader.

Blade graders and multiple-blade drags are used for spreading thin layers of rock asphalt. The material is deposited from trucks in windrows along the center line of the foundation and is then spread over the surface by successive blading until the desired width and depth are obtained. Multiple-blade drags may supplant blade graders entirely or may be used for final planing of the surface to secure a uniform texture and contour.



Courtesy of Kentucky Rock Asphalt Institute.

FIG. 126 Spreading rock asphalt by hand methods.

With hand methods of spreading, firm side forms are set in place to the required width and the rock asphalt is then deposited, preferably on dumping platforms large enough to hold one truckload of material, on an area ahead of that which it is to cover. All lumps are broken up, and the material is shoveled into place with hand shovels. Each shovelful of rock asphalt should be turned over before depositing in order to avoid non-uniformity in the looseness of the material as spread. The rock asphalt is then raked to its full depth by means of hand rakes until it is thoroughly and uniformly loosened and is shaped to the required grade and cross-section. The surface is then smoothed and slightly compacted by means of wooden lutes having a surface about 4 in. wide and 4 ft long and fitted with wood handles and braces. Greater accuracy in securing the required depth, and surface contour, is obtained by placing 14- to 18-ft steel gauge strips (1/8 to 1/4 in. thick and having a width equal to the depth of the required loose layer) in longitudinal rows about 3 ft apart.

Rock asphalt sometimes is heated to facilitate handling or to assist in spreading and compacting in cool or cold weather. Heating is preferably performed in a revolving, cylindrical drier but also is performed by injecting steam into the material in cars or piles.

Compaction of Rock Asphalt Surfaces. Rock asphalt wearing courses are compacted by rolling, the procedure varying to some extent in accordance with the kind of rock asphalt which is involved.

Kentucky rock asphalt contains a very soft bitumen (see Chapter VI, Table 3) which will not permit it to compact until some of the volatile material has evaporated. Kentucky rock asphalt surfaces, therefore, are generally rolled only once, in a longitudinal direction, immediately after placing, with a 10-ton, three-wheel roller. The pavement gradually loses volatile material and becomes harder; traffic completes the compaction, leaving the pavement firm and smooth. Uvalde rock asphalt surfaces are generally rolled initially with a 5-ton, three-wheel roller and are finally compacted with an 8-ton or 10-ton tandem roller. Heavy three-wheel and tandem rollers are used for rolling Alabama rock asphalt.

MAINTENANCE

The maintenance of rock asphalt wearing courses involves essentially the same operations required for a sheet asphalt or fine-graded asphaltic concrete pavement which were described in Chapter XIII.

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CHAPTER XV

BLOCK PAVEMENTS

The stone-block pavement is believed to be the oldest type of paving. Both stone-block and brick pavements were used in Europe in ancient times. There appears to be some question relative to the first brick pavement in the United States; Charleston, West Virginia, constructed a brick pavement in 1871, and Bloomington, Illinois, claims to have constructed one in 1870. The first rural brick road was built in 1895 in Cuyahoga County, Ohio. Wood-block pavements were first constructed in the United States in New York, Boston, and Philadelphia, about 1840. Round cedar-block pavements were later constructed in Chicago, Detroit, and other central western cities. The early destruction of these wood-block pavements by decay led to the use of chemically treated blocks. The first treated blocks were dipped, and it was not until 1896 that creosoted blocks were prepared by modern processes. Asphalt blocks have been used for paving purposes since about 1890. Rubber blocks and cast-iron blocks have been tried on a few experimental paving projects.

The details of the construction of block pavements vary with each of the types of blocks, but the essential operations involved comprise: (1) the construction of a foundation or base course, (2) the construction of a bedding course or cushion course, (3) the placing of the blocks, and (4) filling the joints.

Foundations for Block Pavements. Concrete foundations or base courses are most widely favored for all block pavements because they bridge over slight inequalities in subgrades and provide for uniform, firm support for the individual blocks. Gravel, limerock, and broken-stone foundations also are used, especially for brick pavements. Several typical cross-sections of block pavements on concrete foundations are shown in Fig. 116. The design and construction of foundations or base courses have been described in previous chapters and will not be further discussed in this chapter.

Bedding Courses and Joint-Filling Materials. The principal function of bedding or cushion courses is to distribute the load from the blocks as uniformly as possible to the foundation or base course. Slight irregularities occur in the surface of foundations and in some types of blocks themselves, and a stable support for the blocks and a true surface contour of the finished pavement are secured by firmly placing the blocks so that their upper surfaces are at the elevation of the finished pavement surface. The most common materials for bedding courses are: sand, bituminous mastic (a mixture of sand and bituminous material), granulated slag, dry Portland-cement mortar, damp Portland-cement mortar, and tar pitch.

The materials for filling the joints between the individual blocks provide lateral support for the blocks and should prevent water from entering and damaging the pavement. In addition, they may provide for expansion and contraction of the blocks.

The use of different bedding courses and joint fillers for each of the various types of block pavements has developed as the result of characteristics which are peculiar to each kind of block, and the reasons for each selection will be discussed in connection with the description of each type of pavement.

BRICK PAVEMENTS

When properly designed and constructed, a brick pavement is very satisfactory for heavy traffic; it has a long life, is highly resistant to wear, sanitary, easy-riding, and pleasing in appearance The original cost of brick pavements has been their principal disadvantage but has been reduced to some extent by the use of relatively thin bricks.

Bedding Courses and Joint Fillers. Paving bricks have slight variations in thickness, and the bedding course adjusts these variations and the variations in elevation of the foundation course, and provides firm support for the bricks The various materials used for bedding courses include sand, granulated slag, dry cement mortar, and bituminous mastic. Sand has been most widely used and is satisfactory if thoroughly and evenly compacted and completely confined. Breaks in the foundation or openings between the edges of the pavement and the curbs or gutters may permit water to enter and displace the sand cushion. Granulated slag, which possesses cementing properties, is a satisfactory bedding course material but is economically available in a limited number of locations Dry cement mortar was once used quite extensively but now is seldom used because cracks in foundation courses tend to be transmitted through the bedding course and because it interferes with expansion and contraction of the bricks. The bituminous mastic cushion is gaining in popularity because it is stable, practically waterproof, and has some ability to bridge over cracks in concrete foundations.

Sand has been used to a limited extent for filling the joints but is entirely unsatisfactory because it is not waterproof and is easily displaced. Portland-cement grout was extensively used when steel-tired vehicles were numerous and it was necessary to protect the edges of the brick from wear. Grout fillers crack and do not provide for expansion and contraction, and now are used to a much lesser extent than in former years. Bituminous fillers now are generally preferred because they prevent displacement of the bricks, are waterproof, and provide for expansion and contraction of the brick.

Materials for Brick Pavements

Raw Materials for Brick. Shale and clay are the raw materials for the manufacture of paving brick, the clay being of the fireclay variety. The clay, or the shale after being ground to proper size, must be plastic when mixed with water. In addition, it must be of such a nature as to resist shrinkage, warping, and cracking, and must be capable of producing a hard and tough brick.

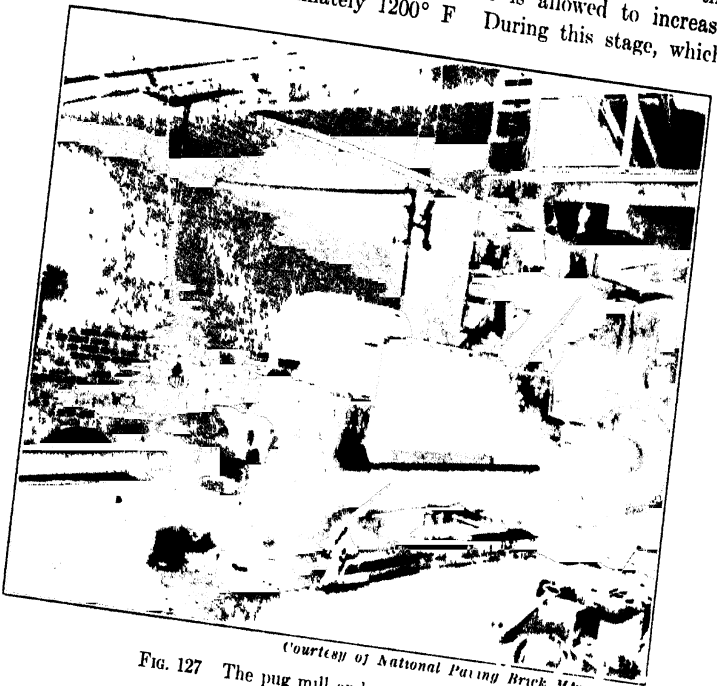
Manufacture of Paving Brick. The manufacture of paving brick involves the following operations: excavation of the clay or shale, crushing and screening the dry material, mixing the water, forming and cutting of the brick, drying, burning, and annealing.

Fireclay is excavated by drift mining operations. Shale usually is excavated by blasting and loading with power shovels, and must be crushed. The clay or crushed shale is screened through a 4- to 12-mesh screen and then goes to the mixer or pug mill where it is thoroughly mixed with water to the desired consistency. The mixture is then dropped into an auger machine, which is an enclosed cylindrical machine containing a worm or auger which forces the clay forward into a conical former. As the mixture advances it is gradually reduced in size until it reaches the die at the end of the former, from which it emerges as a continuous column having a cross-section slightly larger than that of the brick after burning.

The column of clay leaves the former and moves forward over a movable belt to the cutting table where it is cut into brick by means of fine wires. The brick are then removed to the drying room where a large percentage of their contained moisture is removed at low temperatures This process requires from one to three days, and the brick, which are now quite hard, are piled in the kiln.

The kilns may be continuous or intermittent. The brick are piled on edge in the kiln in courses of varying height, depending on the size of the kiln.

The process of burning embraces four distinct stages: (1) drying, (2) oxidizing, (3) vitrifying, and (4) annealing. In the drying stage the heat is applied very slowly so that the remaining water in the brick will be gradually driven off without injury to them. After the water has been driven off the temperature is allowed to increase gradually up to approximately 1200° F. During this stage, which requires from 10 to 60 hours, all vegetable matter in the clay is burned and the iron compounds are oxidized. In the vitrification stage the brick are heated to a red heat (about 2000° F) and held until they are uniformly and thoroughly burned. The kiln is then tightly closed and the brick are annealed by slow cooling, which, depending on the raw material, may require from three days to two weeks. The brick in the upper one or two courses and in several of the lower courses in the kiln are too hard for use as paving, only those in the central portion of the kiln being suitable for that purpose.

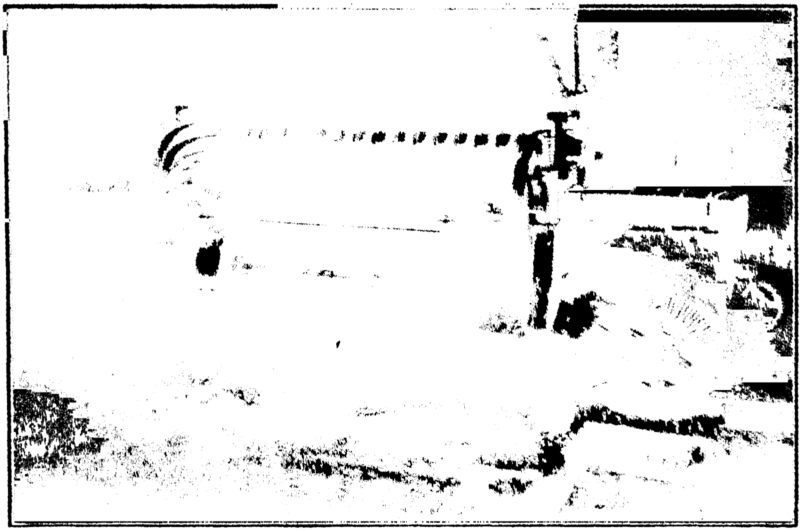


Courtesy of National Paving Brick Mfrs Assoc

FIG. 127 The pug mill and auger machine.

De-airing Clay or Shale. The removal of the air from the clay or shale used for manufacturing paving brick has been practiced by some manufacturers since 1932. The clay or shale is cut or shredded into small pieces after it comes from the pug mill and is then introduced into a vacuum chamber where the air is extracted. The de-airing process is claimed to increase the resistance to wear and the compressive strength, and to reduce the absorption, laminations, and burning shrinkage. The weight of the brick is increased slightly.

Types of Brick. Several types of brick are used for paving purposes, including plain wire-cut brick, wire-cut lug brick, vertical fiber brick, and re-pressed brick.



Courtesy of National Paving Brick Mfrs Assoc.

FIG. 128. Automatic cutting table for cutting the column of clay into bricks by means of small-diameter wires

Plain Wire-Cut Brick. Plain wire-cut brick have straight sides. They possess the advantage that they can be laid on the edge or on the face. They are more readily handled and piled than lug brick but have the disadvantage that complete filling of the joints, as the brick are ordinarily laid, is impossible.

Wire-Cut Lug Brick. The lugs on wire-cut brick are formed by the cutting wires on the cutting table. These wires are guided through slots, which are shaped to form the lugs. The lugs are formed on only one side of the brick. This type of brick is known as the Dunn wire-cut lug brick. Wire-cut lug brick are laid with the smooth and smallest face uppermost.

Vertical-Fiber Brick. As the clay mixture is forced through the conical former it is compressed in a series of spiral layers. The central layers travel forward more rapidly than the outer layers and tend to produce a slightly laminated structure, in which the clay or shale particles probably arrange themselves with their longest dimensions parallel to the axis of the conical former. When wire-cut brick are laid with the wire-cut face upward this structure is therefore vertical in the pavement. The term "vertical-fiber" has been given to wire-cut brick thus laid on account of the type of the brick structure.

The advantage that vertical-fiber brick possess on account of their structure probably is slight, but their slightly roughened surfaces provide a non-skid effect. Vertical-fiber brick are now manufactured with end and side lugs so as to permit thorough penetration of the bituminous filler and complete sealing of the joints. The lugs act as a temporary spacer until the filler is applied and will break down under the force caused by expansion of the pavement.

Re-pressed Brick. Re-pressed brick, as the name implies, are remolded after cutting. The re-pressing operation is performed in a heavy machine, which is constructed with a box mold into which the brick are dropped, and a heavy plunger which fits into the mold and compresses them. The mold is so shaped that in re-pressing the corners of the brick are rounded and lugs are formed.

Re-pressed brick are rapidly being displaced by wire-cut brick as no important advantage is obtained by the re-pressing. In order to receive the blocks readily, the mold of the re-press machine is made slightly longer and wider than the block. The re-pressed brick are therefore reduced in size in only one direction, which causes a complete rearrangement of the particles in the interior of the brick. It has not been shown that this structural change produces a stronger brick.

Standard Varieties and Sizes of Brick. For many years paving brick were manufactured in a large number of sizes. In 1921 the number of sizes was reduced through the cooperation of the Division

TABLE 32

STANDARD VARIETIES AND SIZES OF PAVING BRICK

Variety	Size, inches		
	Depth	Width	Length
Repressed lug	4	3½	8½
Vertical fiber lug	2½	4	8½
Vertical fiber lug	3	4	8½
Vertical fiber lug	3½	4	8½

of Simplified Practice of the U. S. Department of Commerce. Several revisions have since been made, and two varieties and four sizes, shown in Table 32, are now recognized by the Permanent Committee on Simplification of Varieties and Standards for Vitrified Paving Brick of the U. S. Department of Commerce. A tolerance of ⅛ in. in width and depth and ¼ in. in length is allowed by most specifications.

Quality of Paving Brick. Paving brick should be hard and tough, so as to resist the abrasive and impact forces exerted by traffic. The following maximum losses in the standard rattler test are used to control the quality of paving brick.

SIZE OF BRICK TRANSVERSE DIMENSIONS Inches	MAXIMUM LOSS IN RATTLER TEST Per Cent by Weight
2½ × 4	26
3 × 4	24
3½ × 4	22

All brick should be free from deep kiln marks, cracks, and bad chips, should show an evenly burned structure, and should be free from lime, air pockets, and pronounced laminations.

Sand for Sand Bed. The sand for the sand bed should be clean and well graded. All of it should pass a No. 4 sieve, and the clay and silt content (loss by decantation) should not exceed 5 per cent.

Bituminous Mastic Cushion. The bituminous mastic cushion is composed of a mixture of tar or cut-back asphalt and sand, in the following proportions, by volume:

Sand	92 to 95 per cent
Bituminous material	5 to 8 per cent

The sand should comply with the specifications given in Table 33. Rapid-curing liquid asphalts of Grades RC-1, RC-2, and RC-3, and tar of Grade RT-2, described in Chapter VI, are used for the bituminous cement.

TABLE 33

SPECIFICATION FOR SAND FOR BITUMINOUS MASTICS

(American Association of State Highway Officials)

	PER CENT
Clay lumps, loam, organic matter, foreign matter	None
Total passing No. 4 sieve	100
Total passing No 10 sieve	95–100
Passing No. 10 sieve, retained on No. 40 sieve	18– 50
Passing No. 40 sieve, retained on No. 80 sieve	30– 60
Passing No. 80 sieve, retained on No. 200 sieve	15– 40
Total passing No. 200 sieve	0– 5

Dry Cement Mortar Bedding Course. Dry cement mortar bedding courses are composed of standard Portland cement and sand, mixed in the proportions of 1 part of cement to 3 parts of sand, by volume. A satisfactory sand will conform to the requirements shown in Table 34.

TABLE 34

SPECIFICATION FOR SAND FOR PORTLAND-CEMENT MORTAR

(American Association of State Highway Officials)

Passing No. 8 sieve	100 per cent
Passing No. 50 sieve	15 to 40 per cent
Passing No. 100 sieve	0 to 10 per cent
Removed by decantation	0 to 5 per cent
Organic impurities	Colorimetric test shall not produce a color darker than standard color

Asphalt Filler. A blown asphalt is preferable to straight-distilled asphalt for filling the joints when lug brick are used because it has a higher softening point and is less susceptible to temperature changes. The specifications shown in Table 7, Chapter VI, describe a satisfactory filler made from petroleum.

A blown asphalt is not well suited for joint filler when plain, wire-cut brick are used because it congeals rapidly and will not readily penetrate the narrow joints. Plain wire-cut brick are not recommended for brick pavements, but, when used, the filler should consist of a straight-distilled asphalt having a normal penetration of 30 to 50.

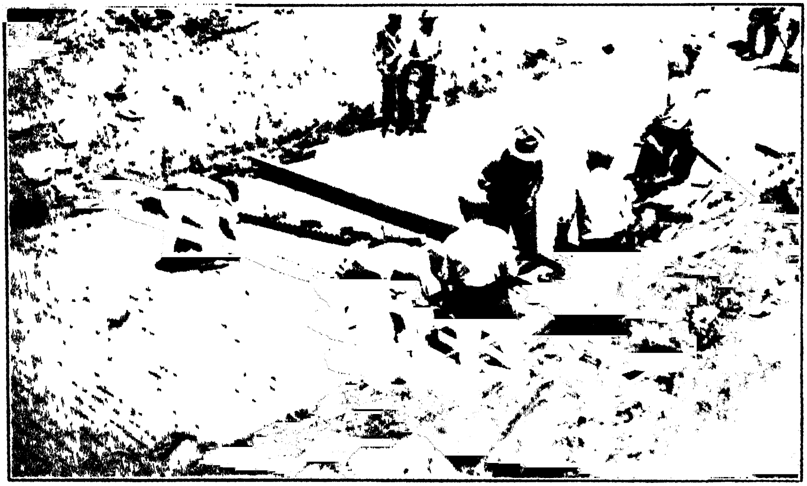
CONSTRUCTION OF BRICK PAVEMENTS

Construction of Bedding Course. The construction of smooth and even base courses for brick pavements makes thick bedding courses unnecessary; a maximum thickness of ¾ in. is sufficient.

When sand, granulated slag, or dry cement mortar is used for the bedding course, the material is spread uniformly on the foundation and struck off by means of a template shaped to the proper cross-section and slightly high to allow for compaction. Compaction is secured by rolling with a hand roller which generally is required to measure not less than 24 in. in width and diameter and to weigh not less than 10 lb per in. of width. Depressions should be filled with additional material and rerolled.

When bituminous mastic is used for the bedding course, the mixture usually is made in a small concrete mixer immediately before being placed. The mastic is spread on the surface, preferably with shovels, and is struck off with a double template which rides on steel forms or on metal tracks on or alongside the curbs. The first template is set slightly above the desired grade, and, after striking off the mastic, the bed is then rolled, usually with a long-handled roller operated by hand. Surface irregularities then are corrected, after which a thin layer of mastic is spread with the second strike board.

Placing the Brick. Brick pavements are laid in straight courses at right angles to the curbs. Each alternate course is started with a half brick in order to prevent the joints in each course from being opposite each other and thereby tending to increase the wear. The bricklayers stand on brick already laid, great care being taken to prevent disturbing the bedding course. The brick are carried to the bricklayers in pallets or clamps, or by means of specially built conveyors. If lug brick are used, they are placed with the lugs all in one direction. The brick are laid so that they are in straight courses and closely touching each other. Driving and ramming, which are practiced in laying stone block, are not permitted with brick. After the brick are laid they are inspected, and objectionable brick are removed by means of flat-nosed tongs and replaced with good ones.

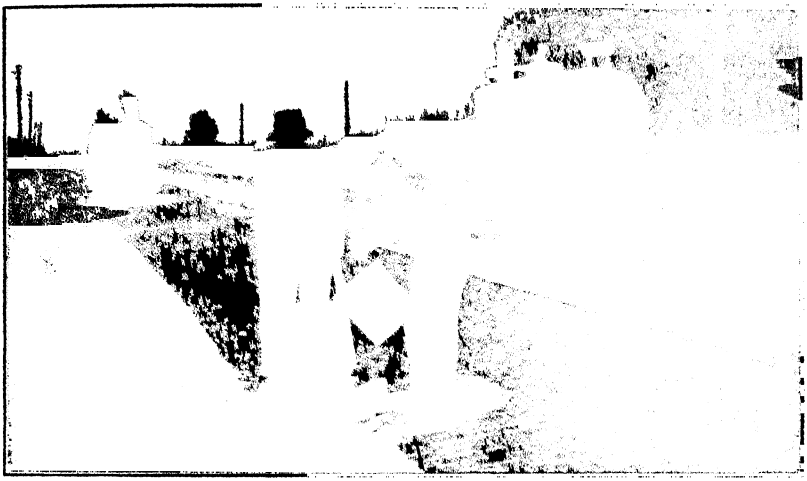


Courtesy of National Paving Brick Mfrs. Assoc.

FIG. 129. Laying brick; note delivery from stockpiles on the shoulders by means of a gravity conveyor.

Rolling the Brick. After placing, the brick are brought to a firm bearing and a true surface by rolling. The usual practice is to roll with a 3- to 5-ton tandem roller, but more recent practice, which originated in Richmond, Virginia, consists of rolling with a 5- to 10-ton tandem roller on boards which have a width of not less than 10 in. and a length of not less than 12 ft, and are dressed to a uniform thickness of $1\frac{3}{16}$ in.; these boards are laid longitudinally on the brick surface and in close contact with each other. The pavement is rolled longitudinally, beginning at the curbs.

After rolling, the brick surface should be checked with a straightedge; the amount of variation should not exceed ⅛ in. in 10 ft.



Courtesy of National Paving Brick Mfrs. Assoc.

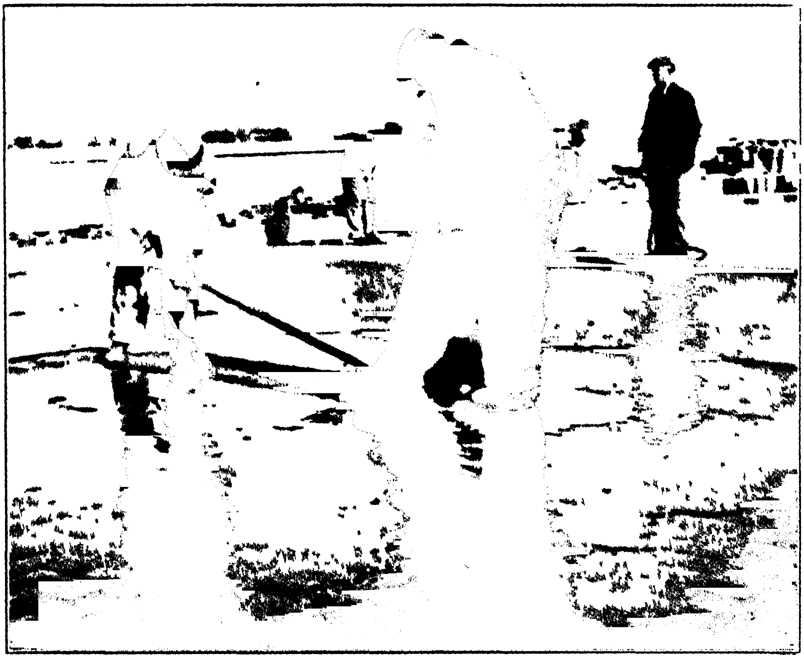
Fig. 130. Application of bituminous filler on a brick pavement by means of squeegee buggy.

When a dry cement mortar bedding course is used, the surface of the pavement is lightly sprinkled with water after the brick has been rolled and checked for surface regularity.

Filling the Joints. The asphalt filler for the joints in brick pavements is usually shipped in steel drums and is heated in kettles to a temperature of 400° to 450° F. Some kettles are designed with warming hoods in which the filler is heated in the drums and flows out one end of the drum into the kettle; in most kettles, however, the filler is removed from the drums by stripping the metal from the asphalt and cutting up the mass of asphalt with axes or a special wire cutter. If the drums are stripped early in the morning, the filler can be cut readily and an entire day's supply placed in kettles.

The joints may be filled by means of a squeegee buggy (see Fig. 130) or the asphalt may be poured directly on the pavement. Some form of squeegee is used if the bituminous material is to be left on the surface of the pavement. The bituminous material is immediately covered with a thin layer of coarse sand, or crushed stone or slag chips, and is rolled. The excess sand or chips may be removed after two or three days.

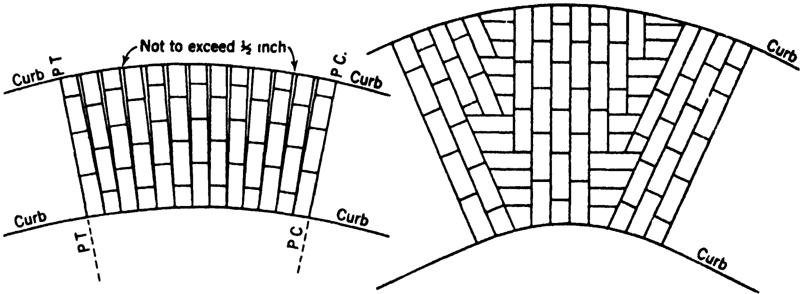
Surface Removal Method of Filler Application. The squeegee method of introducing the bituminous filler into the joint spaces between the bricks leaves a thin film of the bituminous material on top of the bricks. On re-pressed brick, wire-cut lug brick, or plain wire-cut brick laid with smooth faces up, this film does not adhere well enough to remain in place under traffic, and the pavement soon is covered with alternate bare places and covered areas which do not make a pleasing appearance. The bituminous film adheres well to wire-cut surfaces, but when the gritty cover has not been incorporated in the surface asphalt, it has resulted in a surface which is slippery in wet weather. Many engineers prefer to have the surface of the bricks clean—for good appearance in the former case and to have a non-skid surface in the latter. Formerly this result could be secured only by filling the joint spaces with a special type of pouring pot having a



Courtesy of Dept of Public Works, Baltimore

FIG 131 Removing asphalt filler from surface of brick pavement

conical shape, the filler being poured through the pointed end. With these pouring pots the process is slow and costly. The surface-removal method of applying the bituminous filler now is used to obtain a clean surface. In this method the surface of the pavement is moistened and the excess filler is scraped off immediately after placing. Several materials have been tried as the moistening liquid, including whitewash, proprietary solutions, and calcium chloride solutions. A 35 per cent calcium chloride solution containing 1 per cent of starch has been quite successful. The liquid is applied in several ways. In some cases it is applied by means of a padded roller which is kept saturated with the solution. Another successful method consists of spraying the solution through a fine nozzle in the form of a fog; an ordinary orchard sprayer is a satisfactory spraying device. If the nozzle is held vertically over the surface the liquid does not go into the joint spaces as much as when the nozzle is held at an angle with the surface. The small amount of liquid which does trickle down the sides of the brick does no harm because the separating effect is only temporary. The asphalt filler, in this method, is applied while the surface of the brick is wet; it is best to pour the filler on the brick rather than to use a squeegee as a thick coating peels off more easily than a thin layer. The filler should be peeled off as soon as complete settlement has occurred in the joint spaces. Straightened hoes are suitable tools for this work. The material peeled off can be mixed with fresh asphalt and used over again.



(a) (b)

FIG. 132. Methods of laying brick on curves.

Arrangement of Brick on Curves. Bricks may be laid radially on curves if the curvature is slight and the width is not too great, as shown in Fig. 132(*a*). The joint spaces at the outside of the curve should not exceed ½ in. in width. When the curvature and width of the pavement will result in wide joints on the outside of the curve, V-shaped areas (commonly called *dutchmen*), in which bats are employed, are utilized at one or more locations along the curve. Figure 132(*b*) illustrates the use of bat closures. Bats never should be less than 3 in. in length.

Longitudinal Brick Pavements. The standard or conventional pattern of brick pavement construction consists of placing the bricks in transverse courses. The original reason for this practice was the prevention of wear on the edges of the brick by steel-tired vehicles. The Ohio State Highway Department constructed an experimental brick pavement in 1936 in which the bricks were laid with their long dimensions parallel to the longitudinal axis of the road. Several other longitudinal brick pavements have since been constructed. This type of construction appears to be slightly less expensive than transverse construction, owing, probably, to the fact that each brick dropper can walk continuously along the road instead of transversely; more brick droppers can be employed, and the total number of droppers is governed only by the amount of equipment for placing the bedding course, rolling the brick, and placing the joint filler. Longitudinal construction may be somewhat less noisy than transverse construction, and a smooth riding surface is somewhat more difficult to obtain.

Reinforced Brick Pavement. Some experimental pavements have been constructed in which 8 by 8 by 3¾-in. blocks were employed, on a sand bedding course, with ⅜-in. round steel reinforcing bars (placed as a mat) between the blocks and supported 1 in. above the bottoms of the blocks. Joint spaces were filled with 1 : 2 Portland cement mortar. This type of construction, which is still in the experimental stage, may be applicable on stabilized subgrades and on subgrades covered by compacted foundations

MAINTENANCE OF BRICK PAVEMENTS

Brick pavements on concrete foundations, with bituminous mastic cushions and bituminous fillers, generally require very little attention for several years. Shifting of sand cushions causes settlement of the bricks, and the affected areas are repaired by removing the bricks, correcting the cause of the shifting, and replacing the pavement.

The joints in brick pavements occasionally require additional material, and cracks should be filled with bituminous material.

Service cuts require the replacement of the entire pavement, and the back-filled trenches should be thoroughly compacted before replacement of the pavement. Cuts through concrete foundations may be satisfactorily repaired by the methods described in Chapter XII.

Old brick pavements frequently are resurfaced with bituminous wearing courses. Shattered areas should be replaced with concrete, and depressions may be filled with bituminous macadam or coarse-graded asphaltic concrete mixtures.

STONE-BLOCK PAVEMENTS

Stone-block pavements are suitable for very heavily traveled streets in industrial districts; they are constructed almost entirely on concrete foundations. Stone block have not been used on many rural highways because of their high cost in comparison with other types of paving materials.

Bedding Courses and Fillers. Stone blocks are not smooth and regular like other types of blocks, and, therefore, the joints have wider spaces to be filled with filler and the cushion course cannot be smoothed and compacted as for brick pavements. Sand and dry cement mortar are generally used for the bedding course and Portland-cement grout and bituminous mastic for filling the joints. Dry cement mortar usually consists of 1 part of cement and 3 parts of sand, mixed dry. Dry cement mortar bedding courses are somewhat questionable in value with bituminous fillers but are desirable with a Portland-cement grout filler.

MATERIALS FOR STONE-BLOCK PAVEMENTS

Kinds and Quality of Stone. Granite paving blocks are most widely used; sandstone blocks, less frequently. The stone for stone blocks should be of such texture as to enable blocks to be formed without too great an amount of labor and without danger of damaging the blocks during manufacture. The stone should be hard and tough and should wear evenly and without becoming too slippery. Medium-grained stones with a uniform distribution of the constituent minerals are desirable. The American Society for Testing Materials recommends that the following limitations be placed upon the physical properties of granite for paving blocks:

	HEAVY TRAFFIC	MODERATELY HEAVY TRAFFIC
Per cent of wear, Deval abrasion test, not more than	3.6	5
Toughness, not less than	9.0	7

Sandstone should have a percentage of wear of not more than 5.5.

Size of Blocks. Three sizes of stone blocks in common use are as follows:

NAME OF BLOCK	LENGTH ON TOP Inches	WIDTH ON TOP Inches	DEPTH Inches
Standard, or 5-in.	8 to 12	3½ to 4½	4¾ to 5¼
4-in.	7 to 11	4 to 4½	4 to 4½
Hassam	6 to 12	3½ to 4½	4 to 4½

Durax Blocks. A special type of block known as Durax in England and the United States, and as Kleinflaster in Germany, has been used to a considerable extent. These blocks are manufactured from granite and are approximately cubical in shape, their dimensions, in any direction, usually being specified to vary from 3 to 4 in. They should be manufactured from a medium-grained granite having a percentage of wear of 5 or less and a toughness of at least 7. These blocks are manufactured either by a stamping machine or by hand. Their principal advantages include low cost, ease of handling, ease of laying, and the opportunity of selecting the upper face from six faces.

Materials for Bedding Courses and Fillers. The sand for a sand cushion, for a bituminous mastic cushion and joint filler, for a cement mortar cushion, and for a cement grout filler should conform to the same requirements as described for brick pavements. Tar is the bituminous material, and material of Grades RT-10, RT-11, and RT-12, described in Chapter VI, is generally employed.

CONSTRUCTION OF STONE-BLOCK PAVEMENTS

Placing the Bedding Course. The bedding course, whether it be of sand or dry cement mortar, is spread rather loosely on the foundation to a thickness of 1 to 2 in.

Laying the Blocks. Stone blocks are laid with their longest dimension at right angles to the curbs or outside edges of the pavement, except at intersections. The pavers should select the blocks carefully so that blocks of as nearly the same width as possible are placed in the same course. The end joints should be in close contact with each other and the transverse joints as close as possible, keeping the courses straight. Transverse joints should not exceed ½ in. in width, after ramming. The blocks in adjoining courses should break joints at least 3 in. in order to prevent undue wear. The method of laying the blocks is very important. The paver uses a special hammer which has a flat blade on one end. This flat blade is used for scooping out, or truing up, the sand bed for each block. After placing the block the paver gently taps the upper surface so as to bed the block and bring it to the same elevation as the adjacent ones.

Ramming the Blocks. After the blocks are laid they are rammed in place by means of a rammer. There are several types of rammers, of which the one-man rammer is perhaps the most popular. The blocks should be rammed to a solid bearing. With one-man rammers, the number of rammers should be in the proportion of one rammer to each three pavers. Ramming is very important, and the results should be carefully inspected. All blocks which are not solidly bedded or which are low should be removed, the bedding course corrected, and the blocks reset and again rammed. Under no circumstances should blocks be removed by pinch-bars, as they tend to displace the adjacent blocks. Defective or improperly placed blocks may best be removed by means of flat-faced paving tongs.

Before the joints are filled all chipped or otherwise unsatisfactory blocks should be removed and replaced with satisfactory blocks.

Filling the Joints. When a sand-cement bed or Portland-cement grout filler is used (and they are ordinarily used together) the blocks should be lightly sprinkled just before filling the joints. Sufficient water merely to moisten the blocks should be added.

Portland-cement grout may be mixed in special grout mixers or in grout boxes. The usual proportions are 1 part of cement to 1 part of sand with sufficient water to produce a consistency like that of thin cream. The grout is poured into the joints by means of a spout, from a mechanical mixer or large scoop shovels if grouting boxes are used. The grout should be swept into the joints with push brooms. The grout will settle in the joints, and additional grout should be swept in until all the joints are permanently filled even with the tops of the blocks.

Bituminous fillers are heated in heating kettles. Tar fillers should be brought to a temperature of 250° to 300° F. Asphalt fillers may be heated to temperatures of 300° to 350° F. These fillers are usually squeegeed into the joints.

Mastic fillers may be mixed in mechanical mixers or in wheelbarrows. The sand may be heated in specially designed heaters or on steel plates suspended over a fire. The mastic is dumped on the surface of the blocks and squeegeed into the joints. A thin layer of clean sand is then spread over the pavement.

Protection of Pavements from Traffic. Traffic should not be permitted on cement-grout-filled pavements for ten days, and the pavement should be sprinkled two or three times daily until opened to traffic. Stone-block pavements filled with bituminous or mastic fillers should be allowed to cool to approximately air temperature before traffic is admitted. This will usually require from six to eight hours after filling the joints. After one to three days all excess sand should be swept up and removed from the pavement.

DURAX PAVEMENTS

Durax pavements were first laid in Europe in 1885. The first Durax pavement in the United States was laid in the Brooklyn Navy Yard in 1913.

Cushion Course. A sand-cement cushion, in the proportions of 1 part cement to 1 part sand, averaging about 1 in. in thickness, is usually employed.

Laying the Blocks. The blocks are laid in concentric circular segments, the proper curvature being maintained by means of a curved template. The curvature may vary but is commonly 4 ft long, on a straight line from end to end with a center height of 12 in. The larger blocks are placed at the center or crown of the curve, and the smaller ones at the ends of the curved segments. This method of laying the blocks eliminates any joint parallel to the line of traffic.

Filling the Joints. An asphalt mastic for filling the joints is prepared in the same manner as described in connection with the large granite blocks. Grade RC-2 or Grade RC-3 liquid asphalt are suitable bituminous materials; the sand should conform to the requirements described in Table 33.

Adaptability of Durax Pavements. Durax pavements may replace old and badly worn pavements of other types where the original concrete base is still in good condition. On account of the lower cost of the small blocks and the greater area which they will cover, the construction costs less than that with the standard-size blocks.

MAINTENANCE OF STONE-BLOCK PAVEMENTS

The maintenance of well-constructed stone-block pavements during the first few years has usually been very small. When bituminous-filled joints are used it may occasionally be necessary to go over the pavement and refill small depressions which appear. Portland-cement-filled pavements are subject to cracking, and all cracks should be treated with bituminous material in order to prevent water from reaching and displacing the cushion course.

Recut Blocks. Old granite blocks may be used for new or reconstructed block pavements by recutting the blocks. Standard-size blocks may be cut in two, and the two new faces thus obtained laid uppermost in the pavement. In some instances the blocks are merely trimmed to a smaller size. A considerable saving may often be effected by this method of salvaging old blocks.

WOOD-BLOCK PAVEMENTS

Wood blocks have become practically obsolete for street and road pavements, principally because of their high cost, but also in considerable measure because of the unsatisfactory results secured in many instances. Wood-block pavements are quite slippery in wet weather and have a tendency to bleed; they contract and expand with moisture and temperature changes; and their construction requires considerable skill. These tendencies can be overcome to a large extent by the proper selection of blocks and by careful construction. Slipperiness may be decreased measurably by avoiding too much bituminous material in filling the joints, and by having a coarse, sharp sand for the surface covering. The pitch cushion eliminates the objectionable features of sand cushions by preventing water from entering the pavement and displacing the blocks.

Foundations and Cross-Sections. A concrete foundation is the only satisfactory type of foundation for a wood-block pavement. The upper surface of the concrete foundation should be finished as carefully as for a concrete pavement if a pitch cushion is to be used without first constructing a smooth mortar course.

MATERIALS FOR WOOD-BLOCK PAVEMENTS

Kinds of Wood. Many different kinds of wood have been used for wood blocks, but the greatest success has been obtained with southern pine, tamarack, Norway pine, hemlock, and black gum. Southern pine is now used almost exclusively.

Quality of Blocks. The blocks should be free from unsound, loose, or hollow knots, knot holes, worm holes, shakes, and checks; and should be manufactured with square sides and ends. Strong, sound lumber should be chosen, and it is customary to apply the following density rule in order to eliminate undesirable material: The number of annual rings in the 1 in. which begins 2 in. from the pith of the block shall not be less than 6, measured radially; provided, however, that blocks containing between 5 and 6 rings in this inch shall be accepted if they contain 33½ per cent or more of summer wood. In case the block does not contain the pith, the 1 in. to be used shall begin 1 in. away from the ring that is nearest to the heart of the block. The blocks in each charge shall contain an average of at least

70 per cent of heartwood. No single block shall be accepted which contains less than 50 per cent of heartwood.

Types and Sizes of Blocks. Both plain and lug blocks are used. The length of paving blocks depends on the width of the planks from which they are sawed, but is generally restricted to lengths of 6 to 9 in. The width varies from 3 to 4 in. and the depth from 2½ to 4 in.

Preservatives. Creosote oil, or a mixture of creosote oil and coal tar, is generally employed as the preservative. The action of these materials is principally physiological. The creosote is toxic and thus prevents animal organisms from attacking the wood. A physical action is also accomplished by the preservative inasmuch as the pores of the wood are filled, which increases the resistance to wear and decreases absorption and consequent expansion to a considerable extent.

The quantity of preservative necessary to prevent decay is quite small, but paving blocks are usually treated with a much larger quantity than this in order to increase the resistance to wear and reduce absorption to a low amount. From 16 to 18 lb of preservative per cubic foot of wood is generally specified.

Manufacture of the Blocks. Freshly cut timber is rarely made up into paving blocks. One to three months of air seasoning under suitable conditions is usually given the timber. The blocks are cut from planks of a thickness equal to the desired width of block by gang saws capable of sawing a large number of blocks at one time. The blocks are immediately loaded into steel cages mounted on small cars, and placed in the treating chamber.

A treating chamber consists of a steel cylinder, usually 6 to 8 ft in diameter and from 100 to 160 ft in length. The cylinder is charged with the blocks and is then closed air-tight. The blocks are steamed for a period of two to six hours in order to expand and soften the wood and make the moisture content uniform. This steaming process is applied to both green and seasoned timber. The preservative is forced into the blocks under pressure until the required amount has been injected. A final and sometimes a preliminary vacuum is applied.

Inspection of the Blocks. The only accurate way of determining the amount of preservative used in treating the blocks consists of measuring the preservative at the time of treatment. Modern treating plants are equipped with gauges and thermometers which enable this determination to be readily made.

Cushion Courses and Fillers. Sand, dry-mortar, and pitch cushions are used in connection with wood-block pavements. The tar-pitch or asphalt cushion possesses a number of advantages over the other materials. It is impervious to moisture; it does not cause unevenness of the pavement due to expansion of the blocks; it is easily applied; and it is not excessive in cost. Coal-tar and asphalt fillers are generally used.

Surface-Covering Material. After the joints have been filled the pavement is covered with sand, which should be clean and all of it should pass a No. 4 sieve.

CONSTRUCTION OF WOOD-BLOCK PAVEMENTS

Construction of Mortar Cushion Courses. The thickness of a mortar cushion course should be ½ in. The mortar is mixed in proportions of 1 : 1 or 1 : 2 and is spread in place with templates. Dry mortar is usually preferred because it is easier to work with than wet mortar and it eventually sets up.

Construction of Bituminous Cushion. The application of the bituminous cushion is sometimes preceded by the construction of a smooth mortar course approximately ½ in. in thickness, in order to obtain a perfectly smooth base for the bituminous cushion. The mortar course can be eliminated if the surface of the foundation is finished carefully.

The bituminous cushion is applied hot at the rate of 0.3 to 0.4 gal per sq yd. Squeegees are most satisfactory for this purpose.

Placing the Blocks. If a bituminous cushion is used the blocks should be placed within thirty minutes after application of the bituminous material. Wood paving blocks are placed like other types of paving blocks, with their longest dimensions at right angles to the curbs or edges of the pavement. End joints are broken at least 2½ in. If lug blocks are not used the blocks should be separated slightly to provide for expansion. Corrugated paper strips, approximately 1 in. in width, are convenient for this purpose. The blocks are usually set in place with a hammer and are then rolled with a light roller.

Application of Bituminous Filler. The bituminous filler is usually applied by means of squeegees. An excess of bituminous material at any place on the surface of the pavement should be avoided. After the application of the filler, a coating of sand is spread over the pavement. This sand should be spread while the filler is still soft and should be approximately ½ in. thick. This sand layer is usually rolled and allowed to remain in place for twenty-four to forty-eight hours, after which the excess sand is removed. The sand not only takes up excess bitumen but adheres to the surface of the blocks and eventually becomes embedded, thereby creating a less slippery surface and one that is more resistant to wear.

MAINTENANCE OF WOOD-BLOCK PAVEMENTS

Wood blocks made from dense lumber do not show appreciable wear over long periods of time. The maintenance of these pavements is concerned principally with such conditions as bleeding, effects of expansion, damage by water, and slipperiness.

Bleeding. For many years bleeding has been one of the most annoying features in connection with wood-block pavements. Bleeding can be eliminated by proper attention to the selection of the wood, the treatment, and the laying of the pavement. Weiss has summarized the principal features which reduce bleeding as follows:

1. Only slow-grown wood should be used.
2. Green and seasoned timber may be used, but the latter should be steamed.
3. A strong preliminary and final vacuum should be drawn.
4. If tar is used, blocks should be slightly steamed after injection.
5. The blocks should be completely penetrated.
6. The treatment should not exceed 16 lb of oil per cubic foot of wood.
7. Creosote-coal-tar solutions should contain only small percentages of carbon-free tar.
8. The blocks should not be laid too close together.
9. An excess of bitumen should not be used in filling the joints.

When bleeding occurs on a wood-block pavement the only remedy consists in applying a sand covering to take up the excess bitumen. This covering should be removed and replaced as necessary. The sand is to some extent ground into the upper fibers of the wood and tends to seal the blocks and make them less slippery and more resistant to wear.

Expansion. The creosote treatment reduces, but does not eliminate, the absorption of water by the blocks. Expansion and contraction of individual blocks due to both moisture content and temperature changes cannot, therefore, be eliminated, and should be provided for in the design and construction of the pavement. Lug blocks automatically provide for expansion and contraction. Lugless blocks should be separated by means of corrugated paper strips.

Damage by Water. Wood-block pavements constructed on sand beds or with sand filler have given trouble because of washouts. The application of the bituminous paint coat and the bituminous filler practically eliminates this difficulty.

Slipperiness. Wood-block pavements are very slippery under some conditions, particularly during a light, misty rain. Some cities have had more difficulty from slipperiness than others, the difference being attributed to climate, relative humidity, kind of wood, and other conditions. A light sanding of the surface of the pavement will eliminate slipperiness to a large extent, and the effect will be lasting if the treatment is renewed at intervals and traffic allowed to grind the sand into the surface of the blocks.

ASPHALT-BLOCK PAVEMENTS

Asphalt blocks are used principally for street pavements in large cities, for bridge and viaduct roadways, for pier decks, and for industrial floors, roads, and driveways. Large areas of asphalt block pavements have been constructed in New York City and throughout the Metropolitan District; asphalt blocks also have been used extensively for floors on highway bridges in New Jersey.

Bedding Courses and Fillers. Asphalt-block pavements are generally constructed by placing on the foundation a sand-cement or bituminous mortar bed which is about ½ in. in thickness and is finished very smooth; the blocks are immediately laid on this bed. The sand-cement mortar is composed of 1 part of Portland cement to 2½ or 3 parts of sand and is mixed with sufficient water to make a well-dampened mixture. The bituminous mortar may be either a mastic of sand and approximately 5 per cent of cut-back asphalt, by volume, or a cold-mixed and cold-laid mixture composed of sand, mineral filler, and asphaltic material, in about the same proportions as for paving mixtures. A quick-breaking emulsified asphalt is used for filling the joints of street, bridge, and highway pavements, or other surfaces exposed to the weather. The joints of protected surfaces are treated with a slow-breaking emulsified asphalt.

Materials for Asphalt-Block Pavements

Blocks. Asphalt blocks are made with a coarse aggregate, a mineral filler, and asphalt cement.

Coarse Aggregate. The coarse aggregate consists of a hard rock crushed rather fine (at least 97 per cent passing a ¼-in. screen). Crushed traprock, dolomitic limestone, and copper conglomerate (from which the copper has been extracted) are used, the selection being based on quality and availability.

Mineral Filler. Pulverized limestone serves as the fine mineral matter and is usually pulverized to the extent that at least 90 per cent passes the 30-mesh sieve and at least 50 per cent passes the 200-mesh sieve.

Asphalt Cement. A somewhat harder grade of asphalt paving cement is used in the manufacture of asphalt paving blocks than in the construction of other asphaltic pavements, such as asphaltic concrete and sheet asphalt. Penetration limits (at 77° F) between 20 and 50 usually are specified, and for all ordinary street and highway traffic asphalt cement having a penetration of 30 to 35 generally is used.

Proportions of Constituents. Specifications commonly provide that asphalt paving blocks conform to the following composition:

	Per Cen
Retained on ¼-in. screen, not more than	3
Passing ¼-in. screen and retained on 20-mesh sieve	35 to 60
Passing 20-mesh and retained on 100-mesh sieve	15 to 30
Passing 100-mesh sieve, total	20 to 40
Passing 200-mesh sieve, not less than	15
Bitumen	6.5 to 10

Size of Blocks. Asphalt blocks are made in the sizes shown in Table 35. The 12-by-5-in. blocks are standard for streets, highways, bridges, and industrial roadways; the 8-by-4-in. size is used under lighter traffic conditions for resurfacing, for docks and piers, or where construction conditions necessitate a relatively thin block; it also is widely used for floor and roof pavements and for the protection of waterproofing on highway and railroad bridges and grade separation structures.

TABLE 35

Sizes of Asphalt Paving Blocks

Length, inches	Width, inches	Depth, inches	Weight, inches
12	5	3	16
12	5	2½	13½
12	5	2	11
8	4	1½	4¼
8	4	1¼	3½

Manufacture of Blocks. The essential features of an asphalt-block manufacturing plant include an aggregate heater, a mixer, and a hydraulic press. In addition, a plant may be equipped with a crushing plant for crushing its own rock and a pulverizing plant for producing mineral filler. The heating and mixing equipment is of a heavier and more expensive type than that for the preparation of other asphaltic paving mixtures. The heater is much larger and therefore makes possible very close temperature control. The hydraulic press operates at a rapid rate and compresses the mixture at a high pressure as it comes from the mixer. The blocks are cooled by water as they are conveyed from the press.

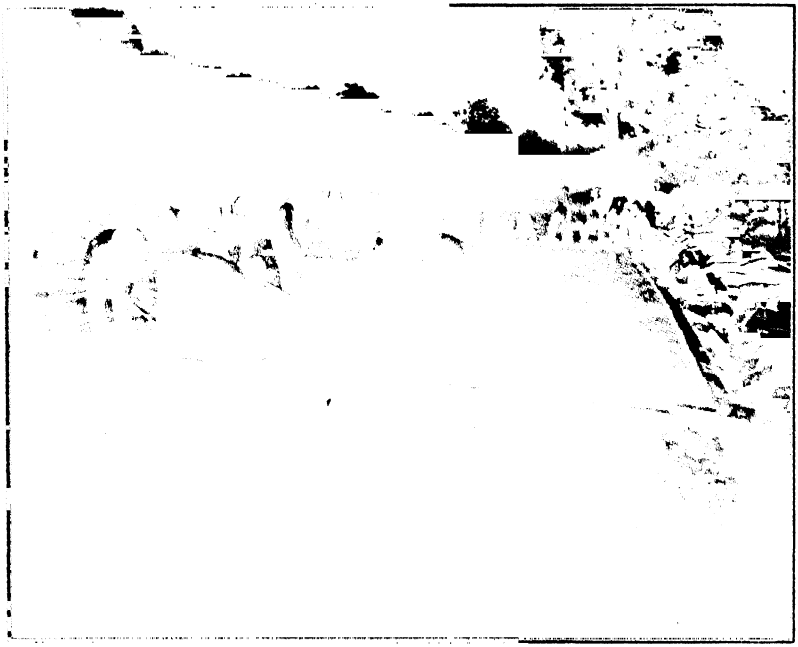
Quality of Blocks. The quality of asphalt blocks usually is controlled by specifying the composition and limiting the absorption to not more than 0.50 per cent. There is also a density requirement, and resistance to abrasion in a rattler of the Talbot-Jones type is occasionally specified.

Materials for Bedding Course and Filler. The cement mortar bed is composed of standard Portland cement and sand; the sand should conform to the specifications shown in Table 34. The quick-breaking emulsified asphalt described in the first column of Table 6, Chapter VI, is a suitable product for filling the joints of exposed pavements.

CONSTRUCTION OF ASPHALT-BLOCK PAVEMENTS

Mortar Bed. The sand-cement or bituminous mortar is spread evenly on the foundation. Steel strips about 4 in. wide and ¼ in. deep are then embedded in the mortar so that their upper surfaces are the depth of the blocks below the finished surface of the pavement. A slight allowance usually is made for compaction of the mortar bed. These strips are set in parallel transverse rows at a distance apart about 1 ft less than the length of the straightedge used for striking off the bed. The surface of the mortar bed is then struck off with a steel straightedge, usually 11 or 16 ft in length, using a short, sweeping motion. The steel strips are then removed and the spaces which they occupied are filled with mortar which is smoothed off by means of a short board strike. The bituminous mortar of the surfacing type of mixture is usually consolidated by rolling.

Laying the Blocks. The blocks are placed on the mortar bed as soon as possible after it has been struck off or rolled to grade. Three rows of blocks are usually laid simultaneously by each paver. The first row is started with a whole block. If the blocks are the 12-by-5-in. size, an 8-in. length is used to start the second row and a 4-in. length to start the third row. When "eight-fours" are used, each alternate course, or row, is started with a half-block. This practice breaks the joints by 4 in., which tends to eliminate wear along the joints. The pavers stand on the blocks already laid when standard blocks are being placed. When "eight-fours" are used, boards are placed over the freshly laid blocks for the pavers to walk on. The blocks are usually carried to the pavers by laborers. The standard-sized blocks are tamped lightly with another block to bed them firmly. A light wooden tamper, having an area of approximately 2 sq ft, is generally used for bedding the "eight-four" blocks. The rows are kept in careful alignment at right angles to the center line of the pavement.



Courtesy of Hastings Pavement Co.

FIG. 133. Placing and striking mortar bed on concrete foundation for asphalt-block pavement.

Filling the Joints. After the blocks are laid, the asphalt emulsion is squeegeed into the joints, keeping the surfaces of the blocks as free as possible from surplus emulsion. A thin covering of clean, fine sand then is broomed over the surface and allowed to remain until the joints are filled with emulsion and sand worked in by the action of traffic.

Protection of Pavement. When a sand-cement mortar bed is used, the pavement should be protected from traffic for at least seven days to permit hardening of the mortar.

Non-Skid Construction. A special method of construction of asphalt-block pavements is followed on grades, curves, intersections, and other locations where a "non-skid" surface is desired. The blocks are placed on the mortar bed, one row at a time, and spacing strips, ⅜ by 2 in. in size, are set on edge between each two adjacent transverse rows. After the blocks are placed, the strips are removed and the joint spaces filled nearly to the top with a free-flowing grout containing 1 part of Portland cement and 1½ parts of sand, by volume. In about half an hour after pouring the grout and before it has taken its initial set, the joints are raked to a depth of not less than ⅜ in. with a special tool closely fitted to the joints so as to leave a clear-cut groove. Drainage of the recessed joints is secured by laying adjacent to each gutter a single row of blocks longitudinally with the gutter and curb and depressed about ½ in. to the raked top of the joints. The pavement should be thoroughly wetted about twice a day for seven or eight days in order to allow the grout to cure properly. If high-early-strength Portland cement is used in the grout, the time required for hardening of the grout may be reduced to about forty-eight hours.

MAINTENANCE OF ASPHALT-BLOCK PAVEMENTS

The maintenance of asphalt-block pavements is not essentially different from that of other block pavements. Damaged blocks are removed and replaced, and joint-filling material is replenished as required. These pavements may be resurfaced by bituminous surface treatments when surface wear or weathering has caused surface deterioration.

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CHAPTER XVI

HIGHWAY AND STREET MAINTENANCE

Highway maintenance consists of the repair and upkeep of

1. the subgrade and shoulders,
2. the roadway,
3. ditches, gutters, and drains,
4. bridges, culverts, and retaining walls,
5. guard rails,
6. signs,
7. traffic stripes,
8. trees and shrubbery, and
9. sidewalks;

together with the work of

10. mowing and removal of weeds and brush,
11. cleaning of pavements,
12. snow removal,
13. inspection of road conditions.

Attention has been called to the fact that, in the performance of maintenance operations on any rural highway system, a large amount of work is done that is not maintenance but improvement. The term "improvement" should apply to work and installations that add to the capital investment in a highway and are not merely items of the upkeep and repair of existing portions of a highway. Thus, the installation of a new driveway culvert is an improvement item, whereas the replacement of an existing driveway culvert which has become defective for any reason is an item of maintenance. Similarly the bituminous surface treatment of a waterbound macadam road or the resurfacing of a gravel road is a maintenance item, but the placing of a 2-in. bituminous road-mix surface on an existing surface-treated, waterbound macadam road is essentially an improvement item. The distinction between improvement and maintenance sometimes is not clearly defined, but it is desirable to separate the two items as accurately as practicable in order to keep reasonably accurate records of cost and in order to assist in the preparation of annual budgets. This condition, of course, occurs most frequently on unimproved or semi-improved local roads and to a lesser extent on improved local roads and state highways. Many local roads are improved on a piecemeal plan, particularly with reference to drainage, through necessity, and, for the reasons stated, it is desirable to keep a separate record of improvements and of maintenance or repairs.

Street maintenance may embrace the same items as rural highway maintenance, although the extent of some items may vary considerably from that on rural highways.

MAINTENANCE OPERATIONS

Subgrade and Shoulders. The shoulders of improved roads require periodical attention to secure drainage of the road surface to the ditches or gutters and to protect the edges of the road surface or pavement from breakage or excessive wear. Earth or gravel shoulders may be reshaped with a blade grader of either the hauled or self-propelled type. When the maintenance of shoulders on paved roads is difficult because of traffic turn-offs or because of the nature of the soil, surfacing of the shoulders with gravel, stabilized soil mixtures, or broken stone is often performed. Oiling also is used in some locations.

The correction of defective subgrades often is a major item of maintenance, particularly in areas subject to frost heave. Drainage, replacement of defective soils, and the other methods of stabilizing subgrades which have been previously described, are used to correct unsatisfactory conditions.

The repair of slopes in embankments and cuts due to erosion is sometimes a costly item of maintenance, and may involve merely the filling of eroded areas or the correction of the condition by appropriate methods.

Roadway Maintenance. Methods of maintaining and repairing the various types of roadway surfaces and pavements have been described in preceding chapters.

Maintenance of Ditches, Gutters, and Drains. Side ditches which have a V-shaped cross-section are usually maintained by means of a blade grader. Ditches having a trapezoidal cross-section require hand methods of cleaning. Excessive erosion of ditches frequently is an annoying problem on rural highways, and its elimination may require the construction of paved gutters, pipe drains, or ditch checks, as described in Chapters III and IV.

Pipe drains and catch basins should be inspected frequently and cleaned out before sedimentation has proceeded too far. The problem of keeping pipe drains open frequently requires a large amount of work on a highway system, such as a county road system embracing 1000 or more miles of road. The careful design and construction of drains to prevent silting is desirable and will eliminate costly items of maintenance. Paved approaches, good grades, tight joints in sandy or silty soils, and freedom from sharp bends will do much to prevent silting; in some locations paved ditches instead of pipe drains are desirable. Pipe drains which have become clogged may often be cleaned with a gouge or shovel, connected to hinged rods; this is a slow operation, and frequently it may be necessary to open the drains at several points to facilitate cleaning.

Bridges, Culverts, and Retaining Walls. The routine maintenance work on bridges consists of cleaning floors, drainage openings, shoes, pedestals, and roller nests; removing obstructions in channels; tightening steel members such as laterals, railings, and floor connections; routine repairs to floors, railings, curbs, approach guard rails, and abutments; and minor repainting. Particular attention is required for the proper maintenance of steel bridges to prevent the deterioration of bridge shoes, expansion devices, and connections below the floor level.

Major maintenance items on bridges comprise work that is extensive and include replacements of floors, cleaning and painting steel, the protection of footings, channel changes, bank protection, and numerous other repairs. Replacement of the floors on old bridges often requires strengthening of the structure to accommodate the additional weight of a suitable floor.

The repainting of steel bridges often is neglected on local road systems and, when rusting has progressed to a large extent, it is sometimes questionable, particularly on small bridges, whether the expense of a good job of complete cleaning is justifiable In numerous cases, it may be more economical to let the old structure continue to deteriorate and replace it when deterioration may create a dangerous condition. Long-span bridges should not be allowed to deteriorate by rusting because of the large cost involved in replacement. Rust can be removed by means of scrapers and wire brushes if it is not too extensive. The removal of extensive rust requires a sand blast, but this is an expensive operation and can be eliminated by proper maintenance when needed. Ordinarily, a complete repainting of steel bridges will be required every six to eight years with minor repairs at more frequent intervals.

Guard Rails. The item of guard-rail maintenance on some road systems often presents a difficult problem, particularly on local highway systems embracing roads in all stages of development. The average layman has little appreciation of the amount of work or cost involved in either the repair or proper elimination of guard rails. Well-financed state highway departments have generally been able to handle this problem effectively, but many county highway departments are forced by lack of finances to defer needed repairs, replacements, or elimination for many years.

Signs. Extremely durable sheet metal signs have been developed for use on highways and streets, but repainting and replacements of both signs and supports are very significant items in maintenance budgets. The destruction and injury of signs by hunters and children often result in large items of expense.

The maintenance of signs on many highway systems often is best performed by a special crew of men who devote all their time to the work. Some maintenance work on signs can be performed on the ground, but much of it can best be done in a shop, and it therefore is desirable to have a reserve supply of signs to replace those undergoing repairs. This practice is not possible with direction signs and some special signs.

Mowing. It is desirable to remove weeds, cut high grass, and remove brush as it appears on rural highways, not only from the viewpoint of appearance but also because such operations increase safety and facilitate drainage. Methods of doing this work vary greatly according to the nature of the ground surface. Mowing can be most effectively performed by means of power mowers where ground conditions permit. Power mowers are available which have mowing speeds up to 5 or 6 mi per hr and transportation speeds up to 25 mi per hr; these mowers are equipped with lifting devices for mowing over the tops of curbs and with racks for carrying extra cutting blades. Uneven ground and the presence of narrow ditches, guard railing, and steep side slopes necessitate hand methods of mowing which add greatly to the cost of roadside maintenance. The number of mowings per year will vary according to rainfall and other conditions; generally two to four mowings per season will keep roadsides reasonably well trimmed, but the extent of mowing will depend upon the necessity and upon the financial ability of the highway organization.

Numerous chemical treatments have been developed for the control of weeds, and some results indicate that valuable progress along these lines may be expected.

Traffic Stripes. Painted stripes are an effective aid for night traffic and are especially desirable on curves and hills. A number of state highway departments have developed specially constructed vehicles for doing this work; hand-operated machines are satisfactory if traffic is not heavy.

Trees and Shrubbery. The care of trees along rural highways includes trimming, removal of dead and fallen trees, and protective measures such as painting and the construction of retaining walls and tree wells. Tree trimming, to be effective, requires skilled workmen, and most state highway departments and public utility companies have developed comprehensive instructions to employees engaged in this work.

Cleaning of Pavements. The cleaning of rural highways usually is not a very extensive item of maintenance and generally consists of the removal of soil washed onto pavements by rains, of debris thrown upon them, and of dead animals. In some localities, particularly on land-service roads subjected to very light traffic, the removal of rubbish piles often is expensive. Some communities have solved this problem quite effectively by providing conveniently located public dumping grounds.

Street cleaning is a major item of street maintenance in many cities and is performed by means of sweeping machines, flushing machines, and by hand methods. Motor-driven street sweepers are equipped with rotating brooms and with sprinkling and pick-up attachments, and, of course, are most effective on smooth pavements. Hand work usually is required at certain seasons of the year to supplement machine sweeping, typical instances being the removal of fallen leaves and debris resulting from parades and celebrations. Motor-driven flushing machines may consist of a single truck unit or a truck and semi-trailer, usually of large capacity to avoid the necessity of too-frequent refilling. Flushing pressures of 30 to 60 lb per sq in. are used, and the larger machines are generally equipped with two nozzles on each side, one being located near the center and the other at the front of the machine. The use of flushing machines is dependent upon large supplies of water being available at low cost, and upon sewer systems capable of handling the water and debris. Fire hose for cleaning pavements generally supplements either machine sweeping or machine flushing.

Snow Removal. Snow removal now is an essential feature of rural highway maintenance as well as of municipal streets and, in areas subject to heavy snowfall, requires a large annual expenditure.

The work of snow removal embraces removal of the snow itself and measures for preventing drifts, and properly includes the sanding of icy pavements.

Causes and Prevention of Drifts. Snowdrifts are formed in much the same way as sand bars in a stream; they are the result of a reduction in the velocity of snow-laden winds, which causes the snow, or a portion of it, to be deposited. Prevention of snowdrifts therefore involves the elimination, so far as is physically and economically possible, of the principal causes of reduction in the velocity of winds at or near highways. This reduction in wind velocity may be caused by fences, by hedge rows, by trees, by variations in the ground surface, and by numerous other physical obstacles. Preventive measures on any highway, therefore, necessitate a careful and intelligent survey of the conditions along the highway that are conducive to drifting and the elimination or alleviation of these conditions as far as possible. Some of the measures which may be adopted to prevent drifting consist of the removal of objectionable fences. This cannot be widely undertaken on a large scale, but educational methods looking to a future betterment of conditions may be pursued and only particularly objectionable fences may actually be removed and replaced.

One of the most effective measures for preventing snowdrifts on highways is the intelligent utilization of snow fences. The most satisfactory type of snow fence consists of wire-woven wooden pickets. The pickets usually are 1½ in. by ½ in. by 4 ft in size and spaced 2 in. apart. This fence is obtainable in 50-ft and 100-ft lengths, the former being more convenient to handle. Steel fence posts, usually U-shaped, serve to support the pickets, the fence being fastened to the posts by wire ties. The posts should have bearing plates attached about 14 to 18 in. from the bottom end to resist blowing over from the force of the wind. In heavy-snow regions long posts (7 to 8 ft) are used and the fence is raised once or twice during the winter as drifts build up in height. Posts 6½ ft high are satisfactory for light to moderate snow conditions. Snow fence should be erected at a considerable distance from the roadway, usually 50 to 100 ft, depending upon snow conditions. Proper locations for snow fences can be determined only by a study of drifting conditions along each highway. Careful observation over a period of years will reveal the best locations.

Snow Removal on Rural Highways. Equipment for removing snow from rural highways includes:

1. Trucks equipped with V-plows.
2. Trucks equipped with straight-blade plows.
3. Trucks equipped with scraper blades.
4. Tractors equipped with V-plows.
5. Rotary snow plows.
6. Blade graders.

The proper kind of equipment for snow removal depends upon road and snow conditions, upon funds available, and upon the particular standard of cleaning which may be or must be adopted.

Truck plows are most widely used for snow removal because of their rapid movement. The straight-blade truck plow is ideal if a sufficient number of units is available to operate over the required area while the snow is falling. Straight-blade truck plows may be operated at comparatively high speeds, up to 25 mi per hr, on fairly smooth road surfaces. They clean the snow closer to the road surface than V-plows do. Straight-blade plows often are used after V-plows to clean the snow left on the surface by the V-plows and to push the snow farther out.

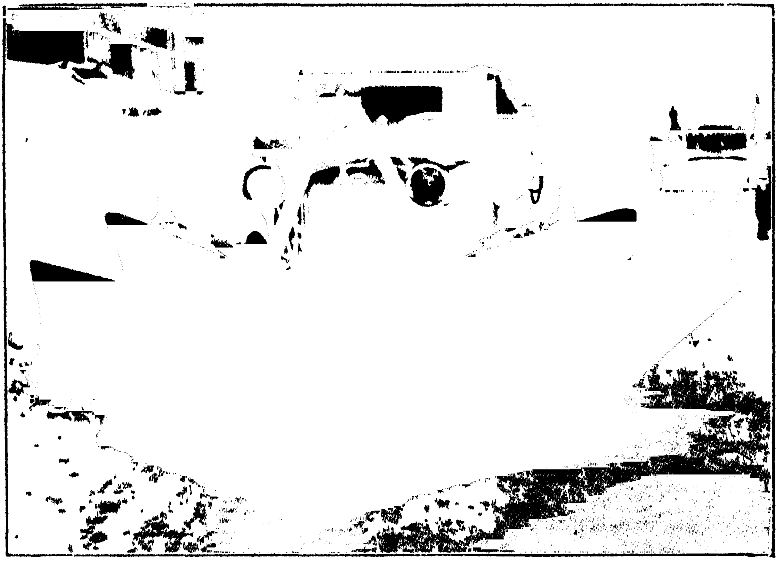
V-plows for use on motor trucks are of two types, the speed plow and the heavy-duty plow. The speed plow is used in place of blade plows and will handle drift conditions more effectively than straight-blade plows. Speed plows are not equipped with wings.

Heavy-duty V-plows, with and without wings, are desirable where heavy snow conditions are encountered. They usually operate at speeds up to 25 mi per hr without the wings in operation Both wings can be used only when snow and traffic conditions will permit, and sometimes only the right wing is used to permit traffic to pass. Hydraulic or air-operated plows are desirable. Sometimes the wings are operated hydraulically, but most operators prefer mechanically operated wings because of their greater strength.

Truck equipment is of great importance in handling snow. Units of 2½ to 5 tons are used most extensively, preference under moderate to heavy conditions being given to trucks of 4- to 5-ton capacity. Four-wheel-drive trucks are used where very heavy snows are encountered In general, large balloon tires are preferable to dual pneumatic tires on rear wheels because they "track" with the front wheels and provide greater traction. Trucks must carry some load as "ballast," and the load should be balanced between front and rear axles; the tendency is toward carrying too much of the load too near the front of the truck.

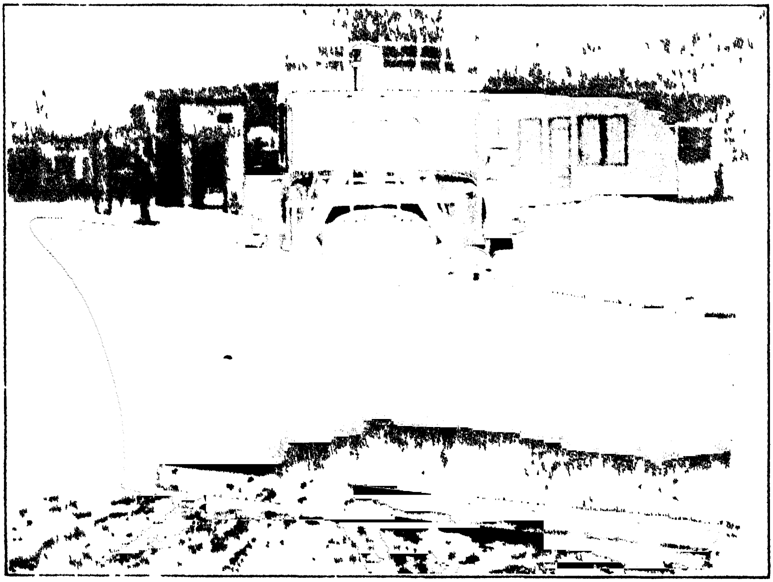
Tractor plows were employed quite extensively before pneumatic-tired trucks came into wide use but have largely been superseded by truck V-plows. Tractor plows are slow in operation but are useful in heavy drifts which cannot be removed by truck plows. Many highway maintenance organizations utilize them as emergency equipment, moving them from place to place on large trailers hauled by trucks.

A rotary snow plow consists of a cutting arrangement ahead of a V-plow which conveys the snow to a fan which throws it at right



Courtesy of Root Spring Scraper Co

Fig. 134. V-type truck snow plow, with wings.

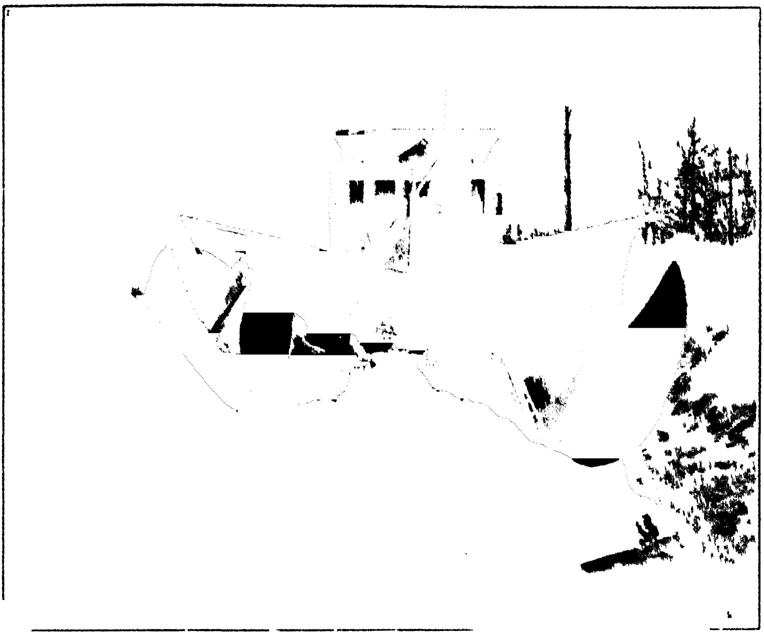


Courtesy of Root Spring Scraper Co

Fig. 135. Straight-blade snow plow mounted on motor truck.

angles to the road. This type of plow is a very expensive piece of equipment and is used for very heavy snow conditions. It will remove snow up to depths of 8 to 10 feet.

Trucks equipped with scraper blades and self-propelled motor graders often are useful for removing light snowfalls and for cleaning the snow left by V-plows.



Courtesy of Penna Dept of Highways.

FIG. 136 Rotary snow plow

Snow removal is essentially an emergency problem, and a carefully trained organization is therefore essential The methods to be followed and the plan of the organization should be worked out well in advance of the first snowfall. Regularly employed maintenance employees should serve as the basis for the snow-removal organization because of their familiarity with topographical and other conditions, but arrangements should be made ahead to have additional forces which may be called into service in times of heavy snowfall.

The equipment should be in good working condition and should be located at conveniently reached points for starting operations. The work of snow removal should be under way by the time the snow has reached a depth of approximately 3 in. and should be in continuous operation until the snowfall has ceased and the roads have been cleared of snow. If truck plows are used their routes should be rather carefully worked out, and each truck should carry two or more laborers with hand shovels to handle short drift conditions which may develop. An essential feature of the organization is provision for moving men and equipment from one section to another, to handle bad drifts or unusually heavy snow. This requires a central office which may be quickly reached by telephone and a system of making frequent reports by the various units of the organization. Radio control has been used to a limited extent and gives promise of usefulness.

Snow removal involves long hours, and is disagreeable and uncomfortable work. Good morale must, therefore, be established in order to keep up a working spirit. Selection of foremen and machine operators from this viewpoint is an important feature of snow-removal organization. Trucks should be equipped with the exhaust type of windshield defrosters, with cab heaters, fire extinguishers, and proper tools to do effective work.

A well-equipped shop, operated by skilled mechanics, and supplied with snow-plow parts, tire chains, extra rear axles, and other truck parts, is essential for effective snow removal.

Snow Removal in Municipalities. Snow removal from municipal streets differs from that on rural highways principally in two respects: preventive measures which may be taken are generally few and are confined to outlying areas in which the streets are essentially rural highways; and methods of removal must generally include provision for immediate or early disposal of the snow.

Snow is generally removed from municipal streets by plowing, by panning, and by flushing. The first method is most common; it consists of plowing the snow into piles or windrows with motor-driven plows, loading it into vehicles, and hauling it to places of disposal. Road graders are frequently used for plowing. Loading is generally performed by hand shoveling, but large quantities of snow are often handled by special snow loaders which load the snow into trucks and trailers. Small tractor plows are useful for removing snow from sidewalks.

Panning snow consists of pushing it into the sewer manholes with large scrapers. This method is usually most efficient in combination with plowing on account of the ease of moving windrows of snow.

Flushing of snow by means of fire hose or flushing machines is an infrequently used method because of the many limitations which affect it.

Methods of snow removal depend upon local conditions relative to disposal locations and upon economical features. Sewers offer the most convenient and economical means of disposal, but their utilization depends upon their gradients, their size, the snowfall, and the normal flow in the sewers. Snow can be dumped directly from vehicles into sewers that have a normal flow of 10 or more cu ft per sec; for smaller sewers and those with light grades, care must be taken in putting the snow into the sewer in order to prevent clogging. In some cases it may be desirable to increase the flow by connecting the water supply with the sewers at a few convenient locations.

Where the sewers cannot be utilized for disposal it is necessary to haul the snow to convenient places of disposal, such as vacant lots, watercourses, or outlying districts where it will not be objectionable. Trailers are efficient in connection with trucks for hauling snow, on account of the light weight of the snow.

Municipalities offer good opportunities for organizing snow-removal forces. The area to be cleaned is more compact than in a rural community, and large numbers of men employed in other municipal departments can be made available for snow removal. Traction and bus companies cooperate in the work of snow removal, and the cooperation of the police department is of great advantage. As in rural communities, the work should be planned well in advance of the snow season, and every responsible member of the snow-removal organization should know his duties and have the necessary equipment in good condition and available for immediate use.

Sanding of Icy Pavements. The safety of traffic requires the sanding of icy pavements at locations of potential danger, such as approaches to highway and railway intersections at grade, curves, hills, and high embankments. A moderately coarse sand is desirable on rural highways; city street departments usually prefer ashes because they are lighter in weight and are more easily carried away by sewer systems. Supplies of sand should be stored at a large number of convenient locations on rural highway systems, and the addition of one sack of calcium chloride per cubic yard of sand will prevent freezing of the sand into large masses which are difficult to handle. Sand is applied by means of hand shovels or by spreaders of the type used for spreading mineral aggregate for surface treatment.

Inspection of Road Conditions. One of the most important features of rural highway maintenance, particularly in connection with local road systems, consists of the frequent inspection of the condition of roads. The ordinary routine of maintenance operations keeps employees in touch with conditions on many roads in a highway system, but it frequently happens that some roads are not seen by maintenance employees for periods of one to several weeks. This is particularly true on land-service roads which are well improved or not improved to any appreciable extent, and may not require extensive maintenance operations. Frequent inspection of roads, especially after storms, will enable maintenance organizations to take prompt care of dangerous conditions which may arise.

ORGANIZATION AND CONTROL OF MAINTENANCE OPERATIONS

The organization of maintenance forces varies greatly, and the arrangement of work, types of equipment, and methods of supervision depend upon the kind of highway system, upon the types of road surfaces, upon topographical features and the layout of the road system, upon financing conditions, and upon many other conditions. Thus, a highway system composed principally of concrete pavements and gravel roads will require a different arrangement for maintenance from one in which bituminous types of pavements and wearing surfaces predominate.

Maintenance Crews. Maintenance operations are very varied in nature, and some items of work are best performed by general maintenance crews while other items are more effectively performed by special crews which devote all, or practically all, their time to the same or similar operations.

The transportation of workmen and the convenient storage of equipment are important considerations, and the division of highway systems into districts is necessary on state highway systems and is desirable on most county highway systems. Area, road mileage, topography, and types of road surface are important factors in the division of a highway system into districts. An average road mileage of 100 to 150 miles generally is sufficient for inclusion in a county maintenance district in moderately populated areas with a variety of road types; many state highway districts contain a lesser mileage on account of the distance of travel. Maintenance of roads in a maintenance district usually is in charge of a district foreman or superintendent.

Special maintenance crews are desirable for performing such work as rail and sign maintenance, extensive patching or resurfacing of pavements, surface treatment, concreting operations, extensive placing or replacing of pipe drains, extensive bridge painting, and other operations. The work of such crews usually embraces more than the area of a maintenance district, and their work can best be assigned and controlled by a general superintendent who has jurisdiction over the entire maintenance program.

General maintenance crews are useful for performing general items of work such as routine patching, floating and grading of surfaces and shoulders, minor bridge and culvert repairs, ditching, cleaning of drains, mowing and removal of weeds and brush, application of calcium chloride, and many similar items.

Maintenance Headquarters. Maintenance yards, equipped with storage garages, and plenty of room for the storage of materials which must be received by rail shipment, are essential for effective maintenance. County maintenance departments should have one or more well-equipped shops, and district yards should have shop facilities for minor repairs to equipment. Railroad facilities are desirable in many cases but frequently need not be provided at all district headquarters.

Many large county highway departments have made effective savings by installing bulk gasoline stations and distributing gasoline to district yards and, in some cases, direct to equipment.

Purchase of Equipment and Materials. Maintenance organizations use a large amount of equipment and large quantities of materials, and effective purchasing is essential to economical operation. Some state laws surround the purchasing of equipment and materials with definite restrictions whereas others confer a large amount of discretionary power on highway officials. It is desirable to purchase many materials in large quantities in order to secure bulk discounts.

Some highway departments install their own equipment for producing sand, gravel, and stone aggregates; others find it equally economical to purchase such materials from commercial producing companies. Unless unusual conditions are present, it usually is desirable to follow the latter practice.

Cost Records. The value of accurate records of the cost of maintenance is obvious, and many highway departments have developed very effective methods of keeping cost records. The principal items of maintenance costs are labor, materials, tools, equipment, and supervision and administration.

Some cost record systems attempt to record maintenance expenditures in too great detail. The necessity of too intricate a classification of costs, in the opinion of the author, is questionable. Highway systems should be divided into a sufficient number of maintenance sections to give a reasonably accurate picture of the cost of maintenance on various parts of the system, but a large amount of detail involves large costs for office work. The author found it convenient, in one county highway department, to record maintenance expenditures, by road sections, under the classifications of annual maintenance and periodic maintenance, the former being subdivided according to labor, materials, equipment, etc., without any attempt to separate the various operations, and the latter being segregated according to the kind of work. Thus, the total cost of ordinary routine maintenance and the detailed cost of such periodic items as surface treatment, bridge and culvert replacement, extensive bridge painting, resurfacing, and other major items, was readily available for comparison and analysis.

Charges for equipment usage may be made to individual road sections by assigning hourly rental rates to individual items of equipment or by spreading the total cost of equipment, including depreciation, as a percentage of the total direct charges, exclusive of equipment. The former procedure is most desirable for accuracy, but it involves more office work and accurate records of each item of equipment must be kept in order to develop accurate rental rates.

Legal Responsibility to Maintain Roads in Safe Condition for Travel. The legal liability of municipalities, townships, counties, and states for the condition of streets and roads becomes important when damage to vehicles, occupants of vehicles, or pedestrians occurs as the result of some defective condition.

It is a general rule of common law that townships, counties, and the state are not liable for damages caused by defective roads unless liability is provided by statute. This rule of no liability is based on the theory that the state is the sovereign power, that townships and counties are divisions of the sovereign power, and that the sovereign power, being supreme, cannot be sued. The desirability of extending relief for damages under some conditions is recognized in some states which provide for a court of claims or a commission of claims to pass on petitions for compensation for damages. The "no liability" rule of common law has been held to apply to municipalities in some states; in others the courts have held that municipalities are liable for the condition of streets on the principle that municipalities are compact corporate units which are created for local benefit and have officers who have authority to act promptly in cases of emergency.

Some states have abrogated the common-law rule of no liability by the enactment of statutes which attempt to define a standard of safe road conditions and describe the conditions under which governmental units become financially responsible for damages sustained. Michigan laws, for example, state that "It is hereby made the duty of townships, villages, cities, or corporations to keep in reasonable repair so that they shall be reasonably safe and convenient for public travel, all public highways, streets, bridges, sidewalks, cross-walks, and culverts that are within their jurisdiction," and provide that negligence shall be conclusively presumed if defects are proved to have existed for a period of thirty days or longer.

The propriety of placing responsibility for damages upon public highway organizations for all highways regardless of the status of their improvement is debatable. Many local highways which are unimproved or semi-improved have some characteristics which might be construed as dangerous or defective, and the amount of money required to eliminate the undesirable conditions would be extremely large; it is very questionable that the emergency powers conferred by statute on public officials are sufficient to enable them to undertake an extensive program of betterments to remove dangerous conditions. The avoidance of liability for damages due to road conditions by the erection and maintenance of appropriate warning signs is provided by statute in some states.

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CHAPTER XVII

SURVEYS, PLANS, SPECIFICATIONS, AND SUPERVISION OF CONSTRUCTION

The term "construction" may properly describe operations which are involved in either highway improvement or highway maintenance. Highway *improvement* generally is understood to refer to additions to the capital investment in a highway, whereas highway *maintenance* refers to the upkeep of the highway, including the replacement of various features. Many highway improvements embrace comparatively simple operations which do not require surveys or elaborate plans, and such improvements often are performed by maintenance employees in the course of their regular duties; examples of such improvements are the construction of additions to a pipe drain, the construction of a short section of guard railing, the construction of a drainage ditch, and the construction of a pipe culvert. On the other hand, some maintenance operations, such as the replacement of a bridge or of a section of pavement, may require surveys and elaborate plans. The discussion which follows refers to complete highway improvements, and it should be understood that the nature and extent of the work of making surveys, preparing plans and specifications, and supervising construction will depend upon the requirements of a given project.

The procedure involved in the complete improvement of a highway from an unimproved or semi-improved to an improved condition embraces:

1. A survey or surveys of the proposed improvement.
2. The preparation of plans and specifications.
3. Financing the improvement.
4. Procurement of needed right-of-way.
5. Award of contract (unless the work will be done by day labor).
6. Supervision of construction.

SURVEYS

The numerous types of surveys made for highway and street purposes include location surveys, surveys for improvement, soil surveys, and construction surveys.

Location Surveys. Location surveys are essential prerequisites for the establishment and improvement of individual highways and for the selection of highway routes. The location of a highway may vary from a comparatively simple procedure for a purely land-service road to a highly complicated study for a section of heavily traveled through-highway. The considerations involved in location are accessibility, adaptability to suitable design standards, safety, and economy.

Location surveys may embrace reconnaissance surveys, preliminary surveys, and final location surveys.

Reconnaissance Surveys. A reconnaissance survey is made for the purpose of selecting the general location of a highway. It may embrace a survey of a large area of unbroken country not already traversed by a highway or it may consist of an examination of several existing roads for the purpose of selecting the location of an entire route or portion of a route; it is made by walking, riding, or flying over the ground and studying the road conditions from the viewpoint of the factors already described. Even when the survey is made by airplane or automobile, some measurements and observations must be made on the ground; necessary equipment usually will include a 50-ft steel tape, a flexible level rod, a compass, a hand level, and an axe.

Preliminary Surveys. Preliminary surveys may be extensive or short and may embrace large areas. They usually are preceded by a reconnaissance survey, particularly when a large area of rugged terrain is involved, but may involve reconnaissance features in themselves. Preliminary surveys may be conducted by running several transit lines and taking cross-section levels, or they may make use of stadia, plane-table, and aerial surveys Their purpose is the securing of sufficient information definitely to locate a highway according to predetermined requirements or to permit the selection of one of several locations. Aerial photographic surveys are particularly adaptable to rough country, over swampy areas, and to congested areas around cities; a specially designed stereoscope makes elevations available for mapping.

Final Location Surveys. Final location surveys are made for the purpose of definitely locating the road on the ground with reference to governmental land lines; they may, and often do, embrace the work necessary for preparing plans for improvement. Many final location surveys, however, are made solely to locate road boundaries, and when this is the purpose they are identical in many respects with land surveys. The boundaries of the road right-of-way, when traffic conditions permit, may best be established by running a transit line which coincides with the center line of the right-of-way; sometimes curves, variations in the width of right-of-way, and the necessity of securing various shapes of property make this procedure impossible. The placing of monuments at road boundaries, along road center lines, or on special reference lines is an important feature of final location surveys. Level surveys are not a necessary part of final location surveys unless information on ground elevations and other elevations is desired.

Surveys for Road Improvements. Surveys for road improvements are made primarily as a basis for design and the preparation of plans; they may embrace an entire road or sections of a road, depending upon the nature and extent of the proposed improvement. They may include all the features of a final location survey, particularly if the initial improvement of a road is involved.

These surveys usually embrace a transit survey, a level survey, and the securing of topographic details.

The Survey Party. The personnel of a survey party varies greatly. On short surveys the party may consist of a surveyor and one or two rodmen. On large surveys several instrumentmen, recorders, chainmen, and rodmen are required, and they may be accompanied by a locating engineer.

Location of the Highway Boundaries. The first work of the survey party consists of the location of the highway boundaries. This often is a simple operation if the highway is located along a section line or monuments defining its boundaries have previously been accurately placed and witnessed. Records establishing the location of roads should always be examined, although many records are defective or not available. In some cases it may be necessary to secure new releases or deeds to establish definitely the location of highways.

The Transit Line. It is desirable to have the transit line coincide with the center line of the traveled surface or pavement or with the center line of the right-of-way; this is not always possible on roads where traffic is heavy, and so an offset line is used.

Staking the Line. Marker stakes usually are placed at regular station (100-ft) intervals along one or two offset lines on either side of the center line; they are used for taking levels, for obtaining topographic details, and for locating various features of the work during construction.

Topographic Details. Complete information relative to the topographic features along the right-of-way is essential to the proper design of the improvement and preparation of plans. The following outline shows the most important details to be observed, measured, and recorded.

1. Government land corners and lines.
2. Property lines and fence lines.
3. Trees, pole lines, shrubbery.
4. Buildings adjacent to the road.
5. Sidewalks and driveways, private roads.
6. Parks and cemeteries.
7. Watercourses (streams, open drains, lakes, etc.).
8. Bridges and culverts; type, span, width, waterway areas, condition, direction of flow, elevations, profile of bottom of stream or channel for 500 to 1000 ft on either side of the road, information regarding high water and erosion. Sometimes the drainage area of culverts is obtained. Sketches of bridges generally should be made.
9. Railroad crossings; location and angle of crossing, elevations of top and bottom of rails, profile of rail for at least 1000 ft on each side of crossing, description of crossing conditions, including sight conditions, signals, signs, nearby switch or passing tracks.
10. Railroad grade separations; elevations, widths, clearances, type, size, condition, drainage. Sketches are generally needed.
11. Railroad bridges and culverts. Same information obtained for highway bridges and culverts.
12. Pipe drains; location, type of pipe, size, condition, inlets, grade.
13. Miscellaneous highway structures; guard railing, retaining walls, signs, spillways.
14. Stock crossings.
15. Governmental bench-marks.

The Level Survey. The U. S. Geological Survey and the U. S. Coast and Geodetic Survey have established bench-marks in various parts of the United States. The elevations of these bench-marks are referred to mean sea level. Where available, these bench-marks should be used. Some highway organizations have established their own datum. If governmental bench-marks are not available it is the usual practice to assume a datum. Care should be taken to assume a datum which is lower than all elevations along the survey, as otherwise minus elevations would occur. Permanent bench-marks must be established along the right-of-way for use during construction. In level or gently rolling country one bench-mark every quarter mile is sufficient, but in rough country they should be established at more frequent intervals. Bench-marks should also be established in close proximity to locations where bridges, culverts, retaining walls, and other structures are to be built. Points selected for bench-marks should be free from the possibility of damage. Permanent concrete structures are suitable. A heavy spike driven into the base of large trees at or just above the ground line is very frequently used. Where no suitable natural benches are available, iron pipe or iron rods, in lengths of 4 ft or more, may be used. All bench-marks should be accurately described in the notes so that they may be easily located at any time.

Bench levels should be taken before running the cross-section levels. It is quite common practice to check the bench levels at the time of taking the cross-section levels by using each bench-mark as a turning point. This practice is open to the objection that, if errors are made, extensive corrections in the notes may be necessary. For this reason, many engineers prefer to check the bench levels before taking any cross-section levels. The elevations of bench-marks should check within 0.05 ft per mile.

Cross-section levels are taken at every station and at intermediate points where an appreciable change in the cross-section or in the profile occurs. The levels should be taken at sufficient distances on each side of the center line to extend beyond the limits of excavation and embankment. In addition to the cross-section levels, readings should be taken on all intersecting roads and driveways so that a profile may be drawn to aid in the design of the improvement.

Soil Surveys. Surveys of subgrade soils may be made for the purpose of determining the causes of subgrade failures on roads already improved or for the purpose of designing and preparing plans for improvement, and may be extensive or comparatively simple. Methods of making soil surveys and preparing drawings of soil profiles will be found in the *Standards* of the American Association of State Highway Officials and the American Society for Testing Materials.

Surveys for Street Improvements. City streets generally are laid out a long time in advance of improvement, and street monuments are located for future reference. A transit line often is run but may be omitted on straight streets. All features which may affect the design or construction are noted, and cross-section levels are taken at intervals of 50 or 100 ft and at such intermediate cross-sections as the contour of the ground surface requires. Levels are taken on all intersecting streets, on all adjacent sidewalks, at all existing drainage inlets, and for all proposed new drainage facilities.

Construction Surveys. Various surveying operations are necessary prior to and during construction, and their extent depends upon the nature and extent of the construction. On extensive projects, particularly when performed by contract, they include setting stakes for grading, setting stakes for drains, curbing, paving and other structures, cross-sectioning of borrow pits, and miscellaneous staking, checking, and measurement.

Slope stakes generally are set for heavy and medium earthwork; grade stakes commonly are used for light grading. Pipe drains, special ditches, curbing, and pavement require stakes to be set for line and grade. Structures such as culverts, bridges, and retaining walls require stakes to be set from which the outline of the structures is given and elevations may be obtained. Long lines of guard railing sometimes are located with a transit but in many cases can be located from pavements (such as concrete) which have regular and well-defined edges.

Small construction projects may be adapted to comparatively simple staking operations. Earth grades can be formed on comparatively level roads by means of blade graders, for example, by setting a row of light stakes about 100 ft to 200 ft apart on line with the inside edge of each ditch bottom; these stakes are destroyed in making the first cut with the grader.

The details of construction surveys are discussed in standard surveying handbooks and will not be further described here.

PLANS AND SPECIFICATIONS

A complete set of plans for the improvement of a highway includes:

1. A location sketch of the proposed improvement.
2. Typical cross-sections of the proposed improvement.
3. The plan and profile of the road.
4. Earthwork cross-sections.
5. Construction details, such as bridges, culverts, guard rails, curbs, gutters, spillways, joint installations, superelevation and widening of curves, etc.

The various steps in the preparation of a complete set of plans are as follows:

1. Drawing the location sketch.
2. Drawing the plan and profile.
3. Plotting the cross-sections.
4. Establishing the grade line.
5. Preparation of drawings showing construction details and the design of all structures.
6. Field inspection.
7. Completion of the plans.
8. Checking the plans.
9. Preparation of estimate of quantities.

Size of Plans. Most state highway plans are placed on sheets 22 by 36 in. in size, the distances between border lines being 21 by 33½ in., which provides a ½-in. margin on three sides and a 2-in. margin at the left of each sheet for binding. Sheets of this size have been adopted as standard by the U. S. Public Roads Administration.

The Location Sketch. An accurate sketch or map showing the location of a highway improvement is an essential feature of the plans for a project to be constructed by contract because it establishes the location where the work is to be done and provides for a mutual understanding by the parties to the contract. On federal-aid projects the location sketch forms a part of the title page, which usually contains, in addition, the name and number of the project, a legend of conventional signs, an index of the drawings included in the set of plans, and spaces for the signatures and dates of signatures of officials who approve the plans.

The cost of plans and of prints for many highway projects can be reduced to some extent by placing the usual title-page data on the plan and profile sheets, by placing additional information on the title page, or by reducing the size (usually in one direction, only) of the title page. Such procedure often is desirable on plans for many local road projects, particularly when the nature of the improvement makes elaborate plans unnecessary.

The Plan and Profile. The plan of a road generally is drawn to a scale of 1 in. = 100 ft and includes the following information:

1. The center line of construction.
2. The boundaries of the right-of-way.
3. Compass or calculated bearings of tangents.
4. Curve data, including points of beginning and ending, points of intersection of tangents (unless impracticable), the radius and degree of curvature, the center point of the curve, and deflection angles.
5. Station points.
6. Streams, railroads, buildings, fences, pole lines, and other structures on the right-of-way.
7. Existing drainage structures, with description of types and sizes, with notation as to their disposal.
8. Proposed drainage structures, with description of types and sizes, and reference to the structural plans to be used.
9. All other proposed structures, with description and reference to the structural plans to be used.
10. Other details which must be included in the improvement.

When a large amount of detail must be included on the plan a scale of 1 in. = 50 ft, or even a smaller scale, is often used.

The profile, for convenience, is placed on the same sheet with, and directly below, the plan (see Fig. 137). The horizontal scale is, there-

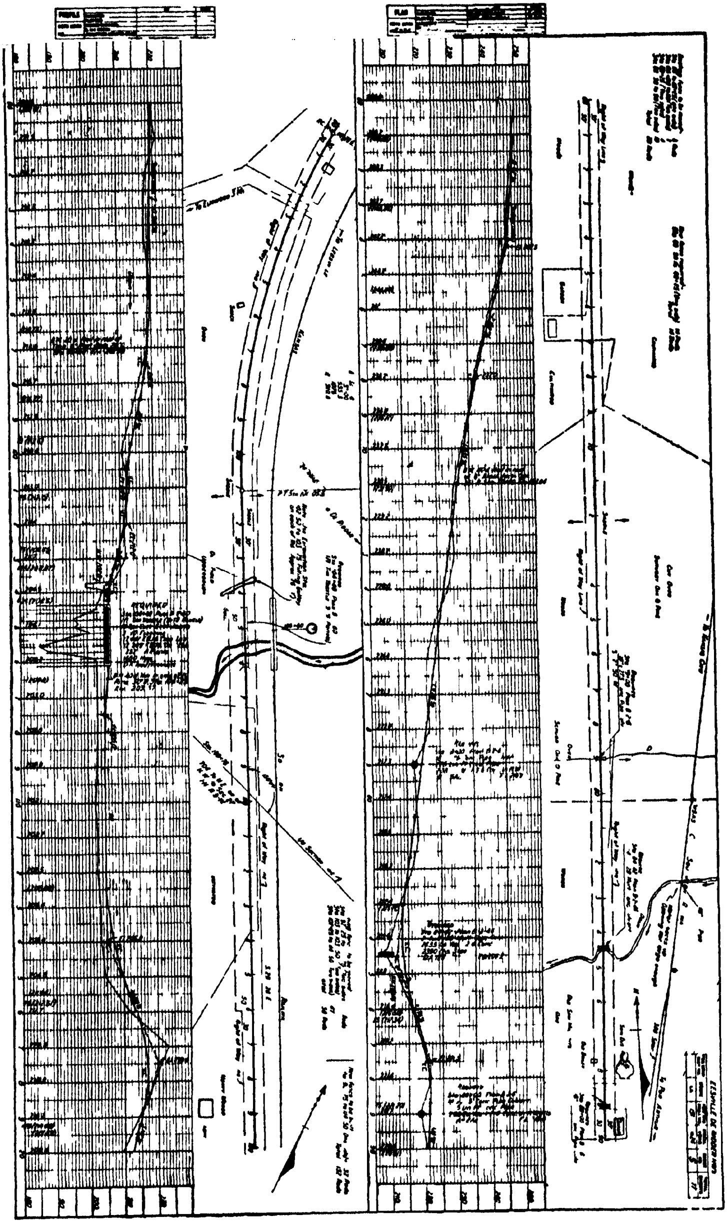


Fig 137 Plan and profile of a highway project

fore, the same as for the plan. A distorted vertical scale is almost always used, a scale of 1 in. to 10 ft being most common. The following information is shown on the profile:

1. Elevation lines, usually marked for each 10 ft.
2. Station ordinate lines.
3. The profile of the existing surface of the old road or of the earth on the center line of construction drawn as a series of connecting straight lines.
4. The grade line of the surface of the finished pavement or surfacing, or the grade line of the subgrade of the improvement, drawn as a series of connecting straight lines.
5. Elevations at points of change in gradient.
6. Vertical curves.
7. The percentage of grade for each continuous grade.
8. The quantities of earthwork, if possible, between balance points of excavation and embankment.
9. The beds of streams and floors of culverts and bridges.
10. Elevations of floors of culverts and bridges.
11. Other special information appertaining to an individual improvement.

The Cross-Sections. The cross-sections of the proposed road are generally drawn to a horizontal and vertical scale of 1 in. to 10 ft. and show:

1. The cross-section of the original, or existing, ground surface.
2. The proposed cross-section of the improvement, exclusive of the pavement or surfacing. (Sometimes the elevation of a balanced flat-top cross-section is used for convenience, in which case the elevation of the top of the cross-section does not coincide with the elevation of the subgrade but bears a definite and constant relation to it.)
3. The location (station reference) of each cross-section.
4. The elevation of the grade line.
5. The quantity of earthwork between adjacent cross-sections.

The cross-sections are used in designing the grade, and the location of the final section, of course, cannot be shown until the grade line is established.

Establishment of the Grade Line. The first step in the establishment of the grade line consists in the determination of the ruling grade for each section of the highway. Each highway organization usually has a definite policy in regard to ruling grades, and the establishment of grades on individual projects is required to conform to this policy.

Three principal conditions are encountered in the establishment of grades, as follows:

1. Locations where the position of the grade line depends upon adjacent topographical features as well as upon economic considerations.
2. Locations where the rates of the existing grades are below the ruling gradient and the new grade must be fitted to the existing grade.
3. Locations where grade reduction is necessary in order to obtain the ruling grade and where economy of grading operation is the controlling factor.

Case I: Grades Suited to Adjacent Topography. Municipal streets offer a good illustration of this case of grade design. The equalization of grading quantities is seldom the controlling factor on city streets; the grade is more usually influenced by drainage conditions, the value of adjacent property, street intersections, sidewalks, and the general appearance of the street. This condition is also frequently encountered on rural highways, particularly in thickly populated sections or in locations where physical structures fix definite points on the grade line. Low and swampy areas and approaches to bridges and overhead grade separation structures generally require the establishment of a grade line which is above the existing ground surface. Under such conditions an excess of either excavation or embankment may occur in such quantity that it will be necessary to waste excavated material which is placed in convenient "spoil banks" or to obtain material for embankments from conveniently located "borrow pits."

Case II: Existing Grades below Ruling Gradient. In those locations where the existing grades are below the ruling grade, the establishment of the grade line is largely a matter of judgment in fitting the new grade to the existing grade so as to obtain a comfortable riding surface, which presents a good appearance with a minimum amount of grading.

Case III: Balanced Grades. In many rural sections in rolling country grade reduction may be effected by balancing quantities of excavation and embankment. Where adjacent property is not seriously affected this method of establishing grades is most economical. The object to be accomplished consists in locating the ruling gradient so that the quantity of excavation will just balance the quantity of embankment with proper allowances for shrinkage.

A practicable method of attack consists of plotting the existing profile and locating trial grades by means of fine thread stretched over the profile and held in place by pins which permit of easy changes without the erasure of penciled lines. The trial grade line should be laid by attempting to balance the cuts and fills visually. Frequent reference must be made to the cross-sections inasmuch as the depth of excavation or embankment on the sides of a highway is often very different from the depth on the center line. After the trial grade has been located, vertical curves should be computed where necessary. An estimate of the quantities of excavation and embankment is then made. These quantities will seldom balance each other on the first trial, and adjustments in the grade must be made until a balance is obtained. The grade lines then are indicated on the profile in pencil.

Quantities of Earthwork. Quantities of earthwork are determined by plotting cross-sections at each station, and at those substations where "breaks" occur in the natural surface of the ground. These cross-sections are usually plotted with the aid of templates. The areas of these cross-sections may be determined by means of a planimeter, although in certain cases the areas may be computed as rapidly as they may be planimetered. Whether computed or determined by means of a planimeter, the area of each cross-section should always be carefully checked.

The volumes of excavation and embankment are usually computed by the average-end-area formula:

$$V = \frac{\frac{1}{2}l(A_1 + A_2)}{27} \tag{56}$$

in which V = the volume, in cubic yards.
A_1 and A_2 = the areas of each of the cross-sections, in square feet.
l = the distance between the cross-sections, in feet (usually 100 ft = 1 station, except where cross-sections at shorter intervals are necessary).

A convenient table for obtaining the volumes of earthwork corresponding to various sums of end-areas of adjacent cross-sections will be found in Appendix B.

Shrinkage of Earthwork. In computing quantities of earthwork an allowance must be made for the shrinkage of an embankment. The term "shrinkage" is perhaps misleading, but is used in lieu of a better term and because of the almost universal use of this designation. The shrinkage factor to be used in any given case will depend upon experience in the particular locality with various soil conditions. The principal condition which influences the correction for shrinkage is perhaps the depth of the excavation and of the embankment. Another important condition is the kind of soil. Vegetation and humus matter in the soil likewise affect the shrinkage, as does also the method of forming the embankment.

The amount of shrinkage must be estimated as accurately as possible, and various highway departments have adopted general rules, which are the result of their experience, for determination of shrinkage The following factors may be said to present average experience:

KIND OF EXCAVATION	EXCESS OF EXCAVATION OVER EMBANKMENT
Light work, with considerable sod	30 to 35 per cent
Light work, without much sod	25 per cent
Medium work	20 per cent
Heavy work	15 per cent

Haul of Earthwork. In excavation work of large magnitude, the length of haul necessary to place materials of excavation in corresponding embankments exerts an important influence upon the cost of the operation. Sometimes it is more economical to waste earth at one location and borrow it for making a fill at another location. The problem in such cases is merely one of economy in which the cost of haul must be compared with the cost of excavation. The quantitative determination of haul of excavation is best accomplished by means of the mass diagram.

The Mass Diagram. A mass diagram is a curve in which the abscissas represent the stations of the survey and the ordinates represent the algebraic sum of excavation and embankment from some point of beginning on the profile. Before calculating the ordinates of the mass diagram, the quantities of embankment involved must be multiplied by their proper shrinkage factors.

Figure 138 shows a mass curve corresponding to the accompanying profile of an existing grade and of a new grade line. If there is both cut and fill at the same station, it is necessary to use the excess of one over the other in computing the ordinate to the mass curve, as the fill is then made from the excavation at the same station, the material being moved at right angles to the center line of the highway, or "cross-hauled." In our consideration of haul we are interested only in the longitudinal movement of the material.

It will be noted from an examination of Fig. 138 that the maximum ordinate of the mass curve (when cuts are considered as plus quantities and fills as minus quantities) occurs at the point where the cut ends and the fill begins; similarly, the minimum ordinate occurs at the point where the fill ends and the cut begins, assuming in each case that we are moving in the direction of stationing.

Inasmuch as the ordinates to a mass curve are plotted from cut-volumes and fill-volumes (fill plus allowance for shrinkage), it follows that the cut and fill between the points at which any horizontal line cuts off a loop of the mass curve will exactly balance Such lines are called "balancing lines" Assuming cuts as plus and fills as minus quantities, loops above a balancing line indicate forward hauling, with reference to the direction of stationing

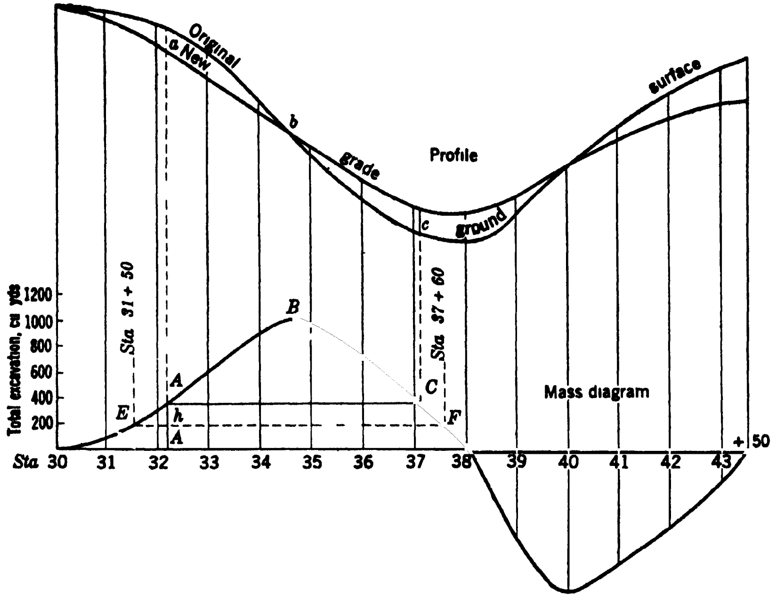


FIG 138 Road profile and accompanying mass diagram

The area between a balancing line and its corresponding loop of the mass curve is a measure of the haul involved in making the cut and fill between the stations represented by the two points which constitute the ends of the loop Thus, in Fig 138 the area between the balancing line *AC* and the loop *ABC* is a measure of the haul performed in making the fill between *b* and *c* from the cut between *a* and *b* Haul is usually measured in terms of the number of cubic yards of material hauled 100 ft, or "station yards"

The number of station yards, or units of haul, in making a fill from a corresponding cut, may be computed from the formula

$$H = \frac{Axy}{100}$$

in which x = the number of feet per inch of abscissa.
y = the number of cubic yards per inch of ordinate.
A = the area between the balancing line and its corresponding loop of the mass curve, in square inches.
H = the haul in station yards.

Overhaul of Earthwork. The price paid for excavation in contract work usually includes any haul that may be necessary. In cases of great variation of haul, however, it is desirable to establish a limit of "free haul," extra payment being provided for hauling all material beyond this limit. This extra distance is known as "overhaul" and must be computed with respect to as much of the material as is affected by it. The most common basis of computation of overhaul is that recommended by the American Railway Engineering Association, which is as follows:

> The limits of free-haul shall be determined by fixing on the profile two points—one on each side of the neutral grade point—one in excavation and the other in embankment, such that the distance between them shall equal the specified free-haul limit and such that the included quantities of excavation and embankment shall balance. All haul on material beyond this free-haul limit shall be estimated and paid for on the basis of the following method of computations, viz.:
>
> All material within the limit of this free-haul shall be eliminated from further consideration.
>
> The distance between the center of gravity of the remaining mass of excavation and the center of gravity of the resulting embankment, less the limit of free-haul as above specified, shall be the overhaul distance.
>
> Overhaul shall be computed in units of 1 cu yd moved 100 ft, and compensation to be rendered therefor shall be computed on such units.
>
> In case material is obtained from borrow pits alongside the embankment and runways constructed, the haul shall be determined by the distance the team necessarily travels. The overhaul shall be determined by multiplying the yardage so hauled by one-half the round trip distance made by the team or vehicle, less the free-haul distance. The runways shall be established by the engineer.

Overhaul may be computed analytically, but its determination may be accomplished more easily by graphical means, making use of the mass diagram. In the case of the profile shown in Fig. 138 a free-haul limit of 500 ft has been established. The overhaul is determined as follows:

1. The line AC is drawn, parallel to the base line of the mass diagram, such that it is 500 ft long, to scale. A is at Station 32 + 20 and C at Station 37 + 20.

2. The centers of gravity of the excavation and resulting embankment are now determined. The point h is half way between A and A',

and the line *EF* is drawn through *h* parallel to the base line of the mass diagram, cutting the mass curve at *E* and *F*. *E* is the center of gravity of the excavation between Station 30 and Station 32 + 20, and, by scale, is at Station 31 + 50. *F* is the center of gravity of the embankment between Station 37 + 20 and Station 38, and, by scale, is at Station 37 + 60.

3. The average haul distance of the excavation from Station 30 + 00 to Station 32 + 20, to make the embankment from Station 37 + 20 to Station 38 + 00, is the distance between the centers of gravity, or 610 ft.

4. The average overhaul distance is equal to the average haul less the free-haul distance, or 610 — 500 ft = 110 ft = 1.10 stations.

5. The overhaul on the material from the excavation between Stations 30 and 32 + 20 is equal to the volume of that material multiplied by its average overhaul distance. The volume is equal to $A'A$, which, by scale, is 360 cu yd. The overhaul is, therefore, 360 × 1.10 = 396 station yards.

Cross-Section Details. One or more sheets are included in the plans to show the cross-sections of wearing courses and foundations, the shape of retaining walls, ditches, curbs and gutters, the details of joints in pavements, the slope of shoulders and roadsides, the details of superelevation and widening of curves, the placing of reinforcement in concrete pavements, the location and cross-sections of sidewalks, the design of intersections, and details of a similar nature. The locations to which these details apply are shown by means of appropriate descriptions.

Structural Drawings. The designs of culverts, bridges, guard rails, retaining walls, and other structures are included in the set of plans. The locations of the structures are often indicated by reference to the station numbers on the plan and profile sheets, or by reference on the latter to the structural drawings.

Field Inspection. After a grade line has been established and the locations and details of cross-sections and structures have been determined, it is the practice of most highway departments to make a careful and thorough inspection of the proposed design on the site of the improvement, using a set of prints of the plans. This inspection is made for the purpose of checking the plans against the actual conditions on the ground. Uncertainties in regard to any details are quickly adjusted, and errors and omissions are readily corrected.

Completion of the Plans. After the field inspection has been made, the plans are completed in conformity with the information obtained from the field inspection. An essential feature of the preparation of plans is the careful checking of all details to see that errors and omissions are eliminated. Some highway departments have devised "check-sheets" which are of assistance in checking the details of design.

Estimate of Quantities. After the plans are completed an estimate of the quantities of each item of work is made, and is expressed in the units which form the basis of payment in contract procedure. These quantities form the basis for the engineer's estimate of cost and for the preparation of proposal forms.

Preparation of Specifications. Contract procedure requires the preparation of very complete specifications for the improvement. Most of the state highway departments and many county and city departments have prepared, in printed form, standard specifications for highway construction. These specifications contain the essential features required on all work, and are supplemented on individual projects by special specifications made necessary by special conditions. This practice is necessary where a large construction program is involved. Typewritten, mimeographed, or multigraphed specifications are used in connection with small construction programs or single projects.

The specifications for a highway project usually consist of two separate parts: the general provisions, dealing with general features such as proposal conditions, the control of the work and materials, the prosecution of the work, and methods of measurement and payment; and the construction details. Sometimes all material requirements are combined in a separate part of the specifications.

FINANCING THE IMPROVEMENT

Preliminary Estimates. Preliminary estimates are sometimes necessary to determine the practicability of an improvement project and sometimes are used as the basis of financing. Necessarily, they are based on incomplete information and should tend to be liberal rather than conservative. Preferably they should be based on a reconnaissance or preliminary survey, and an effort should be made to estimate the nature and extent of the various items of work which will be required.

Engineer's Estimate. The engineer's estimate is based on the estimated quantities of work obtained from the completed plans, and it serves as a basis for financing the improvement and for comparison with contractors' bids. It is made up by assigning unit prices to the various items in the estimate of quantities. These unit prices may be selected by comparison with prices on similar work, received at about the same time; in many cases, however, they should be determined by making a careful cost analysis of each item of work. To the total obtained in this way, there should be added a percentage or amount to cover preliminary expense, engineering expense, and contingencies. Ten per cent is the average percentage for this, but it may be either entirely too high or entirely too low. A typical engineer's estimate is shown in Fig. 139.

ESTIMATE OF COST OF IMPROVEMENT
OF
________________ROAD

Route No.__________ Project No.________ Length 1.74 miles

Location______________________________

Item	Quantity	Unit	Unit Price	Amount
Roadway excavation	7,380	cu yd	$ 0.45	$ 3,321.00
Excavation of old macadam surface	2,050	cu yd	0.75	1,537.50
Borrow excavation	2,360	cu yd	0.55	1,298.00
Concrete pavement	17,830	sq yd	2.10	37,443.00
Brick pavement	3,873	sq yd	3.90	15,104.70
Concrete curb and gutter	2,642	lin ft	0.80	2,113.60
Broken-stone shoulders	3,520	sq yd	0.70	2,464.00
6-in. vit-clay pipe drain	620	lin ft	0.35	217.00
18-in. corr. metal pipe culvert	280	lin ft	2.00	560.00
24-in. corr. metal pipe culvert	124	lin ft	3.00	372.00
Cable guard rail	640	lin ft	0.50	320.00
Concrete, Class A	66	cu yd	20.00	1,320.00
Reinforcing steel	352	lb	0.04	14.08
Sodding	1,335	sq yd	0.30	400.50
Trees removed	3	each	10.00	30.00
Subtotal				$66,515.38
Engineering and contingencies, 10%				6,651.54
Estimated total cost				$73,166.92
Estimated cost per mile				$42,049.90

FIG. 139 Engineer's estimate of cost of improving a rural highway.

Authorization of Improvement. The authorization of an improvement may include the submission of a bonding procedure to popular vote, the authorization and spreading of taxes, the conduct of hearings, and other procedure required by law, or it may involve only a simple order on the part of the administrative official or officials in charge of a highway department. When bonds must be issued or an appropriation made, the amount of money provided should be based on the engineer's estimate. Many miscellaneous improvements are authorized in much the same way as maintenance operations; in fact, they often are included in maintenance budgets for the sake of convenience.

PROCUREMENT OF RIGHT-OF-WAY

All questions in regard to the land areas occupied by a highway should be determined prior to improvement. Right-of-way must be secured for new highways, and laws governing highways usually provide for a very definite procedure to be followed in obtaining right-of-way. The location of the boundaries of a new highway may easily be determined by means of the location survey, but the accurate location of the boundaries of an existing road is sometimes very difficult on account of discrepancies in descriptions in deeds, the absence of accurate records of the establishment of the road, and other conditions. In many cases it may be desirable to draw up new documents which accurately describe the location of the boundaries of existing roads and of changes or additions to the right-of-way.

Purchase of Right-of-Way. The right-of-way for most highways is generally acquired by securing an easement for highway purposes from the property owners, but sometimes is transferred by means of a warranty deed. A form of release of property for highway purposes is shown in Fig. 140.

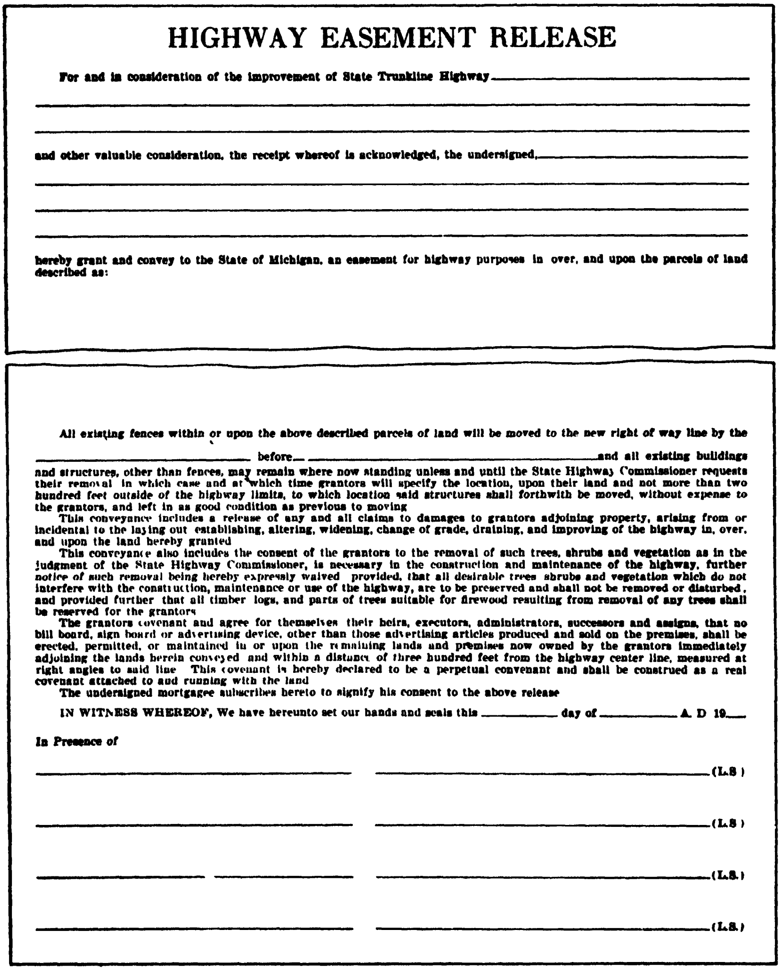
Condemnation of Right-of-Way. When the highway authorities and the owner of land desired for right-of-way are unable to agree upon a price for the property, the land may be condemned by the exercising, on the part of the public, of the right of eminent domain. The condemnation of property is a judicial procedure and is conducted before a court or commission and a judgment is entered. In some states delay in securing property is avoided by legislation enacted to permit work to proceed after the issuance of an executive order, the question of compensation to be decided later.

AWARD OF CONTRACTS

Advertising for Bids. Most state laws provide for public advertisement for proposals for highway improvement, and provide time limits and other regulations for advertising receipt of proposals. The advertisement for bids generally contains the following information:

1. Time and place of receiving and opening proposals.
2. Brief description of location and amount of proposed work.

3. Place where plans, specifications, and detailed information in regard to the work may be obtained.
4. The right to reject any or all proposals.
5. Official signatures of officers in charge of the improvement.



HIGHWAY EASEMENT RELEASE

For and in consideration of the improvement of State Trunkline Highway ____________

and other valuable consideration, the receipt whereof is acknowledged, the undersigned, ____________

hereby grant and convey to the State of Michigan, an easement for highway purposes in over, and upon the parcels of land described as:

All existing fences within or upon the above described parcels of land will be moved to the new right of way line by the ____________ before ____________ and all existing buildings and structures, other than fences, may remain where now standing unless and until the State Highway Commissioner requests their removal in which case and at which time grantors will specify the location, upon their land and not more than two hundred feet outside of the highway limits, to which location said structures shall forthwith be moved, without expense to the grantors, and left in as good condition as previous to moving

This conveyance includes a release of any and all claims to damages to grantors adjoining property, arising from or incidental to the laying out establishing, altering, widening, change of grade, draining, and improving of the highway in, over, and upon the land hereby granted

This conveyance also includes the consent of the grantors to the removal of such trees, shrubs and vegetation as in the judgment of the State Highway Commissioner, is necessary in the construction and maintenance of the highway, further notice of such removal being hereby expressly waived provided, that all desirable trees shrubs and vegetation which do not interfere with the construction, maintenance or use of the highway, are to be preserved and shall not be removed or disturbed, and provided further that all timber logs, and parts of trees suitable for firewood resulting from removal of any trees shall be reserved for the grantors

The grantors covenant and agree for themselves their heirs, executors, administrators, successors and assigns, that no bill board, sign board or advertising device, other than those advertising articles produced and sold on the premises, shall be erected, permitted, or maintained in or upon the remaining lands and premises now owned by the grantors immediately adjoining the lands herein conveyed and within a distance of three hundred feet from the highway center line, measured at right angles to said line This covenant is hereby declared to be a perpetual convenant and shall be construed as a real covenant attached to and running with the land

The undersigned mortgagee subscribes hereto to signify his consent to the above release

IN WITNESS WHEREOF, We have hereunto set our hands and seals this ______ day of ______ A. D 19__

In Presence of

____________ ____________ (L.S)

____________ ____________ (L.S)

____________ ____________ (L.S.)

____________ ____________ (L.S.)

Fig. 140. Form of right-of-way release used by Michigan State Highway Department.

Receipt of Bids and Award of Contract. Laws pertaining to the award of contract generally provide that proposals shall be publicly opened and read. Even where not required, this practice is usually followed. The specifications (sometimes in conformity with statutory provisions) provide for the award of the contract within a specified time after opening proposals and in accordance with a definite policy. In some states, contracts must be awarded to the lowest bidder; in others the public officials are clothed with discretionary power which permits an award to be made to other than the lowest bidder if it can be shown that the lowest bidder is incompetent.

The contract should be as short as possible, consistent with the legal requirements in each instance. Most state laws provide for some form of contract bond to guarantee faithful performance of the contract obligations and payment of all bills for labor, materials, and other expense incurred in prosecuting the work. Because of their impersonal nature, most public departments require, or prefer, bonds executed by surety companies.

SUPERVISION OF CONSTRUCTION

The supervision of the construction of a highway project includes:

1. The establishment and maintenance of detours.
2. Setting stakes for grading, drainage, and structures.
3. Surveys to determine quantities of work done.
4. Inspection of materials.
5. Inspection of construction.
6. The preparation of reports..
7. The preparation of estimates.

Detours. The marking and maintenance of detours usually are done by employees of the public highway organization, and the location of detours when required and arrangements for their maintenance should be made well in advance of construction. The contractor should be responsible for the maintenance of barricades and of warning signs and lights on the project itself.

Staking Operations. Stakes for grading, drainage, and other structures, and the pavement or road surface, are ordinarily set by the engineering force rather than by the contractor's forces. Grading stakes include hub stakes, slope stakes in heavy excavation, and stakes for borrow excavation. Stakes for small drainage structures usually indicate the center line of the roadway and the center line of the structure. The neat lines of abutments, piers, and wing walls of larger structures are usually staked out. Bench-marks should be located sufficiently close to structures to enable elevations to be obtained conveniently with a carpenter's level and a folding rule.

Surveys. If construction operations follow soon after the original survey, a new survey to determine the quantity of excavation may not be necessary. If considerable time has elapsed since the original survey, new cross-section levels are desirable. Changes in the contour of the ground surface frequently occur between the time of the original survey and the beginning of construction, owing to maintenance operations, erosion, and other causes; errors in the original survey also will be detected in the new survey. The engineering personnel which will have charge of the supervision of construction should be on the site considerably in advance of construction to have the construction survey well under way before operations start.

Final surveys of highway projects are desirable to obtain quantities and provide a record of the construction which may be referred to the original plans.

Inspection of Materials. The inspection of materials is an important feature of every construction project and one that is often slighted. There are many materials which must be examined in a laboratory to determine their conformity with specification requirements; these include cement, reinforcing steel, bituminous materials, and many others. Many tests of a simple nature, and requiring no expensive equipment, may be made conveniently and properly on the job. Inspection of manufactured materials at their sources often is desirable. Such materials include mineral aggregates, structural steel, pipe for culverts and drains, timber, and numerous others. Economy, both in construction and inspection, should not be overlooked in connection with the inspection of materials.

Inspection of Construction. The primary purpose of the engineering organization is to obtain a first-class job which is in accordance with the specifications. The engineering personnel on construction usually consists of a construction engineer (generally designated resident engineer or project engineer), an instrument man, and one or more inspectors, the number depending upon the nature and magnitude of the work and upon the extent of the construction operations.

Records and Reports. Records are an important feature of inspection of every construction project; they include survey notes, computations, cross-sections, plan changes, daily records, construction estimates, extra-work records, and miscellaneous reports and records. Most highway organizations have their own printed forms for these various purposes. A daily diary of the progress of construction work is of great value. The work of preparing records and reports can be overdone, and often is, but complete information should be kept daily. Photographic records of construction projects often are desirable for record purposes.

Preparation of Estimates. It is general practice on contract construction to make payment to the contractor for work done at regular intervals, usually once or twice monthly. Payment is based on amounts of work done at the stipulated contract prices, and a percentage is retained until the final completion and acceptance of the work. The amount retained varies according to the policy of the highway department or in accordance with legal requirements, and usually amounts to 10 to 20 per cent of the amount due on each estimate.

Some highway organizations follow the practice of including materials delivered, but not used, in progress estimates to contractors. This practice is quite common and, of course, is helpful to the contractor in financing the work. Care should be taken to pay for materials delivered at proper rates.

The laws of some states require that final contract estimates, and sometimes progress estimates, shall not be paid until the contractor files sworn affidavits that all indebtedness on the contract has been paid in full.

CHAPTER XVIII

ECONOMICS OF HIGHWAY IMPROVEMENT

The highway user is vitally interested in the cost of highway transportation. The organization in charge of the construction, maintenance, and operation of a highway system, therefore, must be familiar with the effect of various road conditions on the cost of transportation over a highway.

COST OF HIGHWAY TRANSPORTATION

The same elements of cost which are encountered in railway transportation are present in highway transportation. These costs may be classified into two groups, road costs and vehicle costs. These two items of cost may be expressed in terms of units of transportation, such as the vehicle-mile, the passenger-mile, or the ton-mile. The first unit is used extensively because of its convenience.

Capital Cost of a Highway. The capital cost or the investment in a highway is composed of the following items:

1. The value of the right-of-way.
2. The cost or value of the highway grade.
3. The cost or value of ditches and drains.
4. The cost or value of bridges and culverts, including grade separation structures.
5. The cost or value of the roadway surface, including bases or foundations.
6. The cost or value of signs, guard railings, and other accessory structures.
7. The cost or value of roadside improvements such as the planting of trees and shrubbery, drinking appurtenances, comfort stations, and similar items.
8. The cost of engineering and administration.

These various items of capital cost, with the exception of the value of the right-of-way, represent the *cost of construction*. When accurate records of expenditures are available, exact data may be used; otherwise the value of each item must be estimated.

The value of the right-of-way usually must be estimated except in instances where completely new locations are involved and expenditures for the land are obtainable. In some cases the value of the right-of-way is estimated as being equal to the value of the adjacent land. In other instances a higher value may quite properly be assigned.

Change in Value. The capital value of a highway fluctuates with the value of money, and if changes in value are to be taken into consideration, the current price-index must be applied to the various items of value.

Reconstruction. If the traffic on a highway never increased beyond a satisfactory and economic limit for the type and design of wearing surface of the original construction, any reconstruction should be classed as an operating expense If, however, reconstruction involves a change in type, in width, in thickness, or in some other fundamental feature of design, at least a portion of the reconstruction cost should be considered as a capital charge; the exact amount will depend upon the salvage value of the original wearing surface.

Operating Costs of a Highway. The operating costs of a highway comprise the following items:

1. Interest on investment.
2. Depreciation.
3. Yearly maintenance costs.
4. Periodic maintenance costs.
5. Engineering and administration.

Interest on Investment. Interest should be included as a part of the annual cost of a highway. Hewes and Glover state that "If a highway were built from cash in the public treasury it would theoretically still be necessary to include in the annual cost of such a highway the interest on the first cost of construction at a rate which the municipality or county could obtain by investment of its funds. The question of how long such interest should run has never been determined. Theoretically, interest should run until the improved road had paid for itself by a saving to the community." Interest rates vary according to location and with economic conditions, but, in general, rates of 2 to 5 per cent may be considered applicable.

Depreciation. The item of depreciation in connection with highway costs offers many opportunities for differences of opinion. Depreciation refers to a condition of gradual destruction or deterioration, and in accounting practice an allowance is made for the amount of this deterioration up to a certain point of time. This theory of depreciation is applicable to certain articles and things, such, for instance, as a piece of machinery, for the reason that it may be more economical to provide for only current repairs or maintenance and allow the machine slowly to wear out rather than to replace its various parts. Such practice is thus based largely on the relative economy of total replacement as compared with partial replacement, and is due to the fact that under existing conditions total replacement by complete manufacture is less expensive than partial replacement involving repair operations. A highway can hardly be considered in this light. It is a public utility, designed and constructed for continuous service. It should therefore be constantly maintained in condition to render such service. The standard of maintenance is of great importance, and provision should be made for the inclusion of replacement items as a part of annual cost. It is manifestly impossible to formulate, with great accuracy, fixed rules for determining the time of replacements, because physical conditions vary to such a great extent. The cost of replacements should be spread as uniformly as possible, however. For purposes of economic comparisons the cost of replacements should preferably be calculated on the basis of a yearly annuity of sufficient amount to provide for replacements.

Yearly Maintenance. The annual maintenance of a highway is the maintenance work that occurs every year. Yearly maintenance may be classified into its various components as follows:

1. Maintenance of ditches and drains
2. Maintenance of bridges and culverts.
3. Maintenance of shoulders.
4. Maintenance of the roadway.
5. Maintenance of accessory structures.
6. Roadside maintenance, including mowing, cutting weeds, removal of brush and trees, and upkeep of other roadside equipment.
7. Snow removal.

Periodic Maintenance. Periodic maintenance consists of various items of work that do not occur every year on the same road. This work includes replacements of surface, bridges, and other structures, and such other items as oiling, surface treatment, and complete reconstruction.

Annual Cost of Highway Operation. The annual cost of highway operation may be expressed or determined by means of the following formula:

$$C_h = rI + M + A_1 + A_2 + O \qquad (57)$$

in which C_h = the annual cost of highway operation.
I = the value of the highway, or the investment.
r = the rate of interest.
M = the yearly expenditure for ordinary repairs which are necessary every year.

A_1 and A_2 = annuities which will provide a sufficient sum of money for periodic maintenance required at intervals of several years.

O = the annual cost of administration.

It may be desirable to subdivide periodic maintenance further, using separate symbols to denote various items such as surface treatment, replacements of surface, and replacements of bridges.

Formula 57, of course, is applicable not only to a single highway, but also to an entire highway system.

The annuities in formula 57 may be expressed in terms of actual expenditures as follows:

$$A_1 = \frac{E_1 r}{(1 + r)^{n_1} - 1} \qquad A_2 = \frac{E_2 r}{(1 + r)^{n_2} - 1}$$

in which E_1 = the expenditure for periodical maintenance of a sort which occurs every n_1 years.

E_2 = the expenditure for periodical maintenance of a sort which occurs every n_2 years.

Salvage Value. Changes in value and changes in capital costs can be taken into consideration in formula 57 by the assignment of proper amounts for the investment item represented by I.

Formula 57 takes depreciation into consideration as an item of periodic replacement (such as A_1, A$_2$, or A_3). If, however, depreciation is considered as an actual loss of value (in other words, if major replacement of the highway is not assumed as periodic maintenance) another item should be introduced into formula 57, namely, *salvage value,* and would be expressed as an annuity, thus

$$\frac{(I - S)r}{(1 + r)^n - 1} \tag{58}$$

in which S = the salvage value of the highway at the end of n years.

The author prefers the first method for purposes of economic comparison when changes in the capital investment are not involved (as, for example, changes in type, width, or thickness).

However, when, for purposes of economic comparison or statistical information, changes in the capital investment are to be considered, the inclusion of a salvage value in formula 57 has a definite advantage because it permits the evaluation of the existing structure as a part of the reconstruction.

Professor C. B. Breed uses the following approximate formula for determining annual road costs, taking salvage value into consideration

$$C_h = \frac{(I + S)r}{2} + \frac{I - S}{n} + M + \frac{E}{n} \tag{59}$$

in which E = the total periodic maintenance cost required during a life of n years. Formula 59 is easier to apply than formula 57, and the results agree closely with those obtained by formula 57.

The accurate evaluation of life and salvage value of highways presents many difficulties. Anson Marston, when Dean of Engineering at Iowa State College, undertook a study of the service lives of street pavements in the city of Des Moines, Iowa, and estimated the average life of pavements from mortality or survivor curves (see Fig. 141).

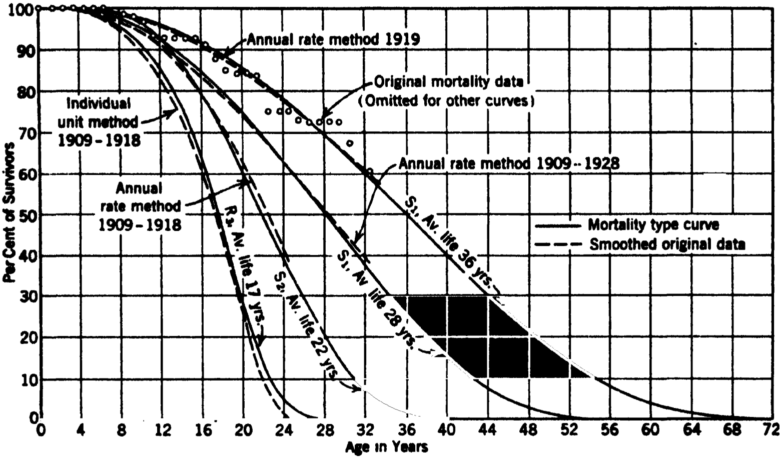


FIG. 141. Mortality curves of brick on concrete pavements, Des Moines, Iowa, 1909–1928. (*Marston.*)

Winfrey and Farrell reported, in 1941, that the average service lives of highway surfaces on the rural portions of primary state highways were as follows:

Soil surfaces	5 to 14 years
Gravel or stone surfaces	6 to 13 years
Bituminous treated surfaces	11 to 21 years
Mixed bituminous surfaces, low-cost types	14 to 22 years
Bituminous macadam surfaces	15 to 17 years
Bituminous concrete surfaces	13 to 20 years
Concrete surfaces	17 to 24 years
Brick and block surfaces	18 to 21 years

Elements of Vehicle Costs. The costs of vehicle operation include the following items:

Fixed costs:

1. Interest.
2. Depreciation.

3. Insurance.
4. Taxes and fees (excluding gas and other sales taxes).
5. Housing.
6. Supervision, if commercial fleets are involved.

Variable costs:

1. Gasoline.
2. Oil and grease.
3. Cleaning.
4. Ordinary repairs and maintenance.
5. Tires.
6. Major repairs and overhauls.
7. Wages for drivers.

Annual Cost of Vehicle Operation. The annual cost of vehicle operation is represented by the following formula:

$$C_v = rI + A + F + M \tag{60}$$

in which C_v = the annual cost of vehicle operation.
I = the original investment.
r = the rate of interest.
A = an annuity sufficient to replace the vehicle at the end of its economic life.
F = the annual fixed charges.
M = an annuity sufficient to cover the cost of maintenance.

Total Cost of Highway Transportation. The total cost of transportation over a single highway or section of highway may be expressed by means of the following formula:

$$C = C_h + NC_v \tag{61}$$

in which C represents the total cost of transportation, N the number of vehicles using the highway per year, and C_h and C_v have the same significance as in formulas 57 and 60. The value of N may represent an average for a single highway, as determined from traffic counts at several points, or may be applied only to sections of a highway where the traffic is uniform.

The total cost of highway transportation on an entire highway system should be calculated as the sum of the costs on various sections.

The unit cost of transportation on a section of highway may be calculated by dividing the total cost by the number of vehicles, or

$$C = \frac{C_h}{NL} + \frac{C_v}{L} \tag{62}$$

in which C represents the cost of transportation per vehicle-mile, and L represents the length of highway in miles.

EFFECT OF ROAD CONDITIONS ON VEHICLE COSTS

Tractive Resistance. The movement of a vehicle along a level highway is resisted by forces within the vehicle itself, caused by the friction of moving parts, by the frictional forces resulting from the contact of the vehicle tires with the highway surface, and by the pressure of the air against the vehicle. The first two sets of forces constitute the *rolling resistance* of the vehicle; the last-named force is the *air resistance* of the vehicle. When the surface of the highway is inclined, the movement of a vehicle is resisted by another force, namely, that required to lift the vehicle through the vertical distance involved. This force is the *grade resistance,* which may be either positive or negative, depending upon the direction of the grade or slope, that is, whether it is up or down. The sum of these forces is called the *tractive resistance* of the vehicle.

The rolling resistance of vehicles varies according to their mechanical characteristics and condition and, of course, according to the kind and condition of the roadway surface. The rolling resistance of a vehicle is usually determined from dynamometer tests and it is approximately proportional to the weight of the vehicle. Present-day passenger vehicles on smooth, firm pavements have rolling resistances equal to about 20 to 25 lb per ton of weight.

The air resistance of vehicles depends upon the area and shape of the vehicle exposed to the pressure of the air and upon the speed, and is expressed by the formula

$$R_a = KAV^n \tag{63}$$

in which R_a = the total air resistance, in pounds.
A = the projected area of the vehicle, in square feet.
V = the speed of the vehicle, in miles per hour.
K and n = experimental coefficients.

Values of K and n are determined from actual tests of the pressures resulting from air blown against the surfaces of vehicles; values of $K = 0.0018$ and $n = 2$ are representative of present-day passenger vehicles.

The grade resistance encountered by a motor vehicle is given by the formula

$$R_g = \pm \frac{Wg}{100}$$

in which R_g = the grade resistance, in pounds.
W = the gross weight of the vehicle, in pounds.
g = the rise or fall of the grade per 100 ft of distance, in feet (equals the per cent of grade).

The grade resistance usually is expressed in pounds per ton of weight. The grade resistance per ton is $\pm \frac{2000g}{100} = \pm 20g$, and therefore

$$R_g = \pm 20Tg \tag{64}$$

When a vehicle is traveling at constant speed, the tractive effort is equal to the tractive resistance; thus

$$E = R = R_r + R_a \pm R_g \tag{65}$$

in which E is the tractive effort, R the tractive resistance, and R_r the rolling resistance. Substituting the values of R_a and R_g from equations 63 and 64, equation 65 becomes

$$E = R = R_r + 0.0018V^2 \pm 20Tg \tag{66}$$

Effect of Type and Condition of Wearing Surface. The type and condition of wearing surface affects principally the item of fuel consumption of motor vehicles, but, on such surfaces as untreated earth, untreated gravel, and untreated traffic-bound macadam, the cost of tire wear, tire repairs, cleaning, mechanical and body repairs, and depreciation may be greatly increased in comparison with that on surfaces which are smooth and practically free from loose fragments and dust. Comparative costs necessarily are subject to great variations in individual instances and data obtained in one area may not apply to another. Studies of rolling plus air resistance show differences between roadway surfaces in similar classifications, but the greatest differences occur, of course, between low-type and high-type surfaces. Professors R. A. Moyer and Robley Winfrey, in 1939, published the results of a study of the cost of operating several hundred vehicles on earth and gravel surfaces and on pavements; the relative costs of operation (including depreciation and other fixed costs) were as follows:

	Annual Mileage, 8000	Annual Mileage, 15,000
Earth surfaces	1.25	1.36
Gravel surfaces	1.18	1.25
Pavements	1.00	1.00

These values agree reasonably well with those obtained from other studies and appear to be accurately representative of these types of surfaces as a whole.

Effect of Grades. The principal effect of grades on the cost of vehicle operation is the increase in fuel consumption in comparison with level grades. The saving in time resulting from the maintenance of a constant speed may be important for heavy commercial vehicles but is not so important for light vehicles. Increases in oil consumption and tire wear are small. Therefore, most economic studies have been concerned with the increased cost of fuel consumption alone. The fuel consumption on grades can best be determined by actual measurement. Such measurements should be made on actual roadways having reasonably long straight distances at uniform rates of grade. Measurements of fuel consumption should be made in still air and by means of accurate instruments.

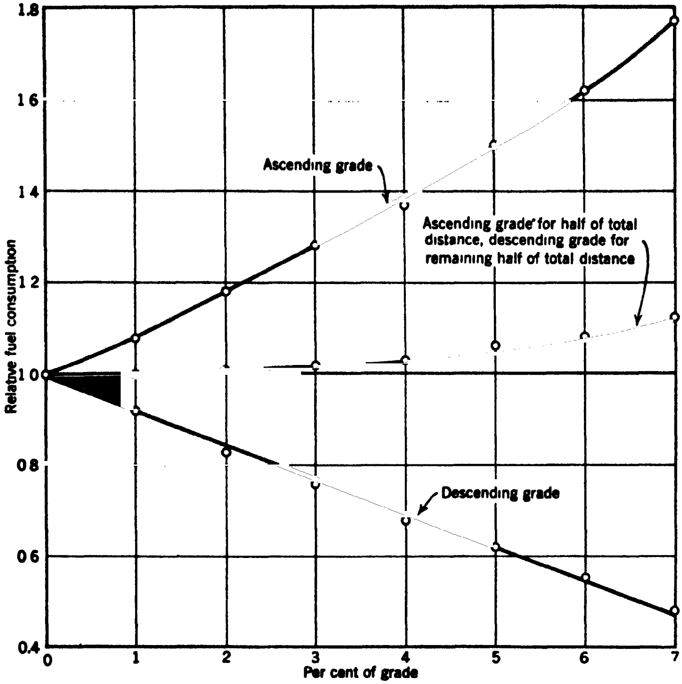


FIG 142 Gasoline consumption of a private passenger automobile traveling at a constant speed on various grades (*Based on test data by Professor John A Oakey*)

The additional fuel required to propel a vehicle up an ascending grade at the same speed as on the level is directly proportional to the rate of grade. The relative fuel consumption of a private passenger vehicle for various rates of ascending grades, under a given set of conditions, is illustrated by the upper diagram of Fig. 142.

Assuming that a vehicle travels at a constant speed, it is apparent that there is a descending grade at which the vehicle will continue to travel at the same speed *without any tractive effort on the part of the* *vehicle motor.* Such a descending grade was defined by Professor H. B. Shaw as a "floating grade." Grades less than the floating grade will require some tractive effort by the vehicle motor. In the ideal case (with the motor shut off), the fuel consumption at the floating grade would be zero, but under actual operating conditions this ideal situation does not obtain. The lower diagram of Fig. 142 shows the fuel consumption, for various descending grades, of the same passenger vehicle to which the upper diagram applies.

It is apparent from Fig. 142 that, for ordinary private passenger vehicles, there is a decrease in fuel consumption for descending grades which compensates to a considerable extent for the increased fuel consumption on equivalent ascending grades. Thus, on highways in rolling or hilly country, the additional fuel consumption in comparison with that on level grades is not great if the total "rise" and "fall" are approximately equal. The effect of rise and fall is illustrated by the intermediate diagram of Fig. 142 which shows the fuel consumption on grades which rise and fall at the same rate for equal distances.

The ideal descending grade for motor vehicles is the floating grade, and numerous studies have shown that the floating grade for most private passenger vehicles is from 6½ to 7½ per cent. The ideal ascending grade for motor vehicles is one which will permit traveling at a constant rate of speed which is safe for the conditions involved and which will require no change of gears. Private passenger vehicles can ascend grades up to 7 per cent in high gear at constant rates of speed of 50 to 60 mi per hr. The fuel consumption of heavy vehicles is affected by grades to a greater extent than that of light vehicles because of their lower ratio of power per ton of gross weight.

The extent to which grade reduction may be practiced on a given highway will depend upon the proportions of the different classes of vehicles comprising the traffic and upon the actual decrease in fuel consumption which may be effected for all the vehicles under the conditions which will exist on that highway after the grade reduction has been accomplished.

Effect of Distance. The unit cost of operating a motor vehicle usually is calculated on the basis of the number of miles the vehicle travels throughout its useful life. Necessarily, the mileage traveled per unit of time affects the unit cost very materially. The total cost is made up, as previously described, of fixed costs and variable costs; the latter are most affected by differences in traveled distance.

Effect of Traffic Delays. Traffic delays are caused by an insufficient number of traffic lanes, by insufficient sight distance on curves, by reduced speed limits on account of municipal regulations, by cross-traffic on intersecting railways, waterways, and other highways, and by a number of miscellaneous conditions. In many cases the delay is sufficiently great that its elimination or reduction is imperative. An economic study is usually desirable in order to determine the amount of the justifiable expenditure. The lost time may be determined by measuring the time consumed and comparing it with the time required to cover the distance under the conditions that are proposed. Examples of methods of reducing delays to traffic are construction of grade separations, changing the time interval of traffic signals at intersections, construction of by-pass highways around municipalities for through-traffic, and widening of highways.

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CHAPTER XIX

HIGHWAY PLANNING, FINANCING, AND ADMINISTRATION

The broad field of highway administration is concerned with the planning, design, improvement, maintenance, financing, and administrative control of highway systems. The subjects of design, improvement, and maintenance have been discussed, and our attention will now be devoted to some of the fundamental considerations involved in the planning, financing, and administrative control of highways.

HIGHWAY PLANNING

The objective of highway planning is the development of a comprehensive, adequate, and economic improvement program and maintenance plan. The early development stage of modern highway improvement in the United States, which covered the period from about 1904 to 1930, was brought about quite suddenly by the introduction and rapid development of motor vehicles, and was marked by such an unusual demand for highway facilities that the formulation of adequate and unified plans for the improvement of highway systems was almost impossible. More recently, it has been possible to develop plans for rehabilitating and extending highway services according to sound engineering, economic, and sociological principles.

The accomplishment of the development of adequate highway systems is made possible only by collecting, compiling, and analyzing information relative to the physical features of these highway systems, the volume, character, and distribution of traffic, and related subjects such as military uses of the highways, the relationship of various forms of transportation, the effect of various conditions (such as agriculture, tourist movements, industry, and others) on highway traffic. The collection of information involves a thorough and careful inventory of highway systems and traffic surveys to determine the essential facts relative to the use of highways.

HIGHWAY INVENTORIES

Road inventories, as the name implies, refer to the accumulation of information in regard to the physical features along highways. Inventory records may be kept in the form of plan and profile sheets, strip maps, general maps, and typewritten records. Plans for improvement, with construction records, may serve as the basis of plan and profile sheets. The following information may be recorded.

1. Plan.
2. Profile.
3. Location of all structures.
4. Cross-sections of wearing surfaces.
5. Buildings.
6. Land lines.
7. Fence lines.
8. Monuments and bench-marks.
9. Watercourses.
10. Soil characteristics and soil profiles.
11. Railroads and railroad crossings.
12. Stock crossings.
13. Snow-fence locations.
14. References to releases and deeds of right-of-way.
15. Sight distances.
16. Special features of local importance.

Information secured from construction drawings and records usually must be supplemented by inventory surveys; automobile odometers are satisfactory for measurements of mileage if checked against known distances; special odometers reading in hundredths of miles are obtainable, but their additional value is somewhat questionable.

Traffic Surveys

Purposes of Traffic Surveys. The principal purposes of traffic surveys may be listed as follows:

1. Determination of priority of improvement.
2. Determination of controlling features of design.
3. Budgeting of funds for construction, reconstruction, and maintenance.
4. Allocation of funds (particularly motor-vehicle funds) to various highway systems.
5. Scheduling of construction and maintenance operations.

Character and Amount of Information Secured. The character and amount of information to be secured in any traffic survey will depend upon the purposes to be served and upon available funds for the work. A traffic survey may be made on one or several roads or streets only or may embrace all or part of the area of a city, metropolitan area, county, or state; in other cases it may be conducted on all or a part of a definite highway system. The distribution of funds (particularly between state and local units of government) and the incorporation of roads into various administrative systems should be, and sometimes is, based on information which is secured from comprehensive statewide traffic surveys.

The basic information desired from almost every traffic survey is the number of vehicles passing a given point in a given length of time, or the density of the traffic. This information can be secured by observation of the traffic with no interference with the movement of vehicles. A traffic survey involving only observation of vehicles often is described as a traffic census. Highway vehicles generally are classified into five or six groups, (1) private passenger cars, (2) trucks, (3) busses, (4) streetcars, (5) motor cycles, and (6) horse-drawn vehicles. In some surveys motor cycles and horse-drawn vehicles are not included. Trucks frequently are subdivided into groups according to size, normal load capacity, type, or legal classification. Trailers sometimes are considered as part of the vehicle to which they are attached and in other cases are listed separately. Additional information which generally is obtained by observation includes the percentage of out-of-state vehicles in each vehicle group; sometimes the direction of travel is noted, and, in some states where license plates bear distinguishing markings, the proportion of local and non-local vehicles is obtained. The number of passengers may be noted, and a general classification of trucks with reference to commodities hauled may also be made.

If more detailed information than that described is desired it is necessary to stop vehicles and interview the drivers. If traffic is heavy, only a percentage of the vehicles is stopped. A postcard, to be filled out by the driver and mailed at destination, often is used to avoid delays to vehicles when the information sought cannot be obtained in a short space of time. This additional information may embrace numerous items related to highway transportation; among the items often secured are origin and destination, purpose of trip (business or pleasure), commodities carried, annual travel mileage, gasoline consumption, and trip time. In some cases vehicles are weighed.

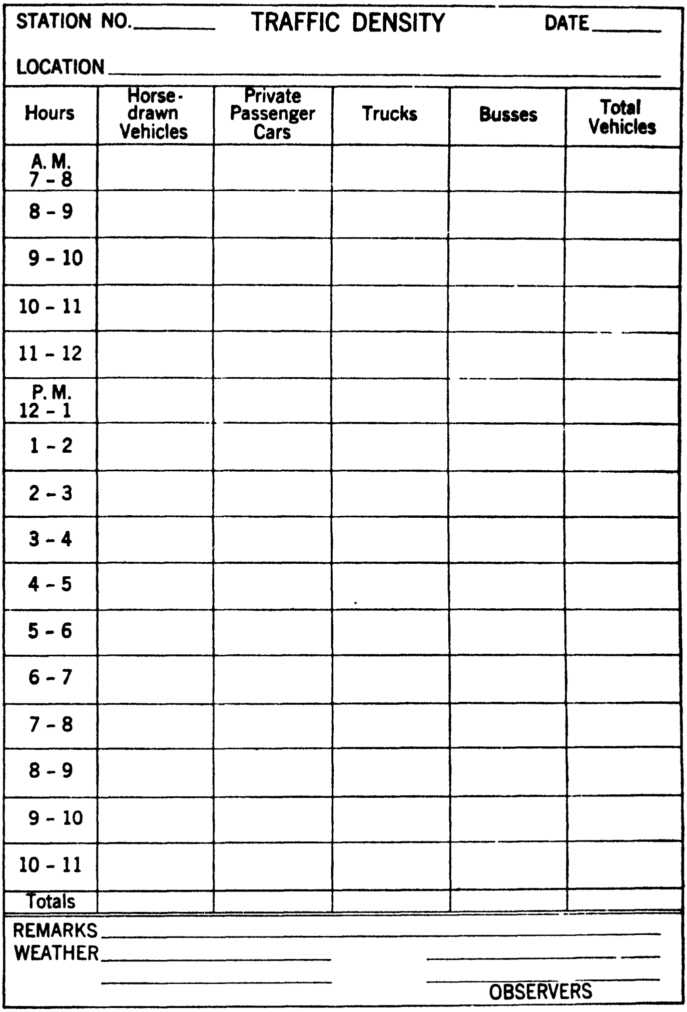
Survey Stations. Survey stations are classified into two groups, (1) primary or control stations, and (2) secondary or limited-count stations. Primary stations may be observation or sampling stations (including weighing stations); generally secondary stations are observation stations.

Survey stations require careful selection, and a knowledge of local conditions is essential to insure accuracy, particularly in the selection of control stations and corresponding secondary stations. Stations generally are selected at intersections because they are locations of changes in traffic and because they offer opportunities of counting traffic on several roads at the same time. Control stations and secondary stations should be established on the basis of the class of road (e.g., interstate routes, main intrastate routes, main county highways, land-service roads, street thoroughfares, residential streets) and the characteristics of the area served; the type of road frequently should be taken into consideration.

Duration of Observations. Traffic counts are usually made for 24-hour days at control stations, divided into three 8-hour periods beginning generally at 6 A.M. Very frequently it will be found that the proportion of vehicles from 10 P.M. to 6 A.M. will be reasonably constant and that the night counts can be greatly reduced or eliminated altogether. Observations at secondary stations usually are made for shorter periods of time, such as 8 hours, 4 hours, or even 1 or 2 hours. The 1-hour or 2-hour periods should not be adopted at a large number of survey stations until the reliability of counts for such short periods has been established.

Frequency of Obervations. The frequency of observations will depend upon the purposes of the survey. If peak traffic volumes are desired, for design or other purposes, observations naturally will be confined to a few days, and a knowledge of local conditions will indicate the days on which observations should be made. If the total yearly traffic (or average daily traffic) is desired, several observations must be made; a three-day census every six or eight weeks will usually be sufficient; it may be desirable to make observations on one or more holidays or on other special days. Three-day observations are made on Saturdays, Sundays, and one weekday; variations in traffic on weekdays from Monday to Friday do occur, but if due regard is paid to weather conditions, and observations are repeated in six or more weeks evenly distributed throughout the year, a reasonably accurate representation of traffic is secured. A few observations generally will establish the effect of adverse weather conditions on traffic volumes within, perhaps, a reasonable degree of accuracy. Gasoline tax collections, particularly if their distribution by counties is available, are useful as a guide to the monthly or seasonal variation in traffic.

Recording Traffic Survey Data. Traffic survey data are recorded usually on printed forms designed to show the information desired. Figure 143 shows a simple form used by the author for recording traffic density on a county highway system by hourly periods for 16-hour daily traffic counts. Figure 144 shows a more elaborate form for recording traffic density on a statewide highway traffic survey. Vehicles usually are recorded by means of a mark for each unit, but hand-operated counting devices sometimes are necessary for heavy traffic. Studies of origin, destination, and ownership of vehicles, nature of



STATION NO.______ TRAFFIC DENSITY DATE______

LOCATION______

Hours	Horse-drawn Vehicles	Private Passenger Cars	Trucks	Busses	Total Vehicles
A. M. 7 - 8					
8 - 9					
9 - 10					
10 - 11					
11 - 12					
P. M. 12 - 1					
1 - 2					
2 - 3					
3 - 4					
4 - 5					
5 - 6					
6 - 7					
7 - 8					
8 - 9					
9 - 10					
10 - 11					
Totals					

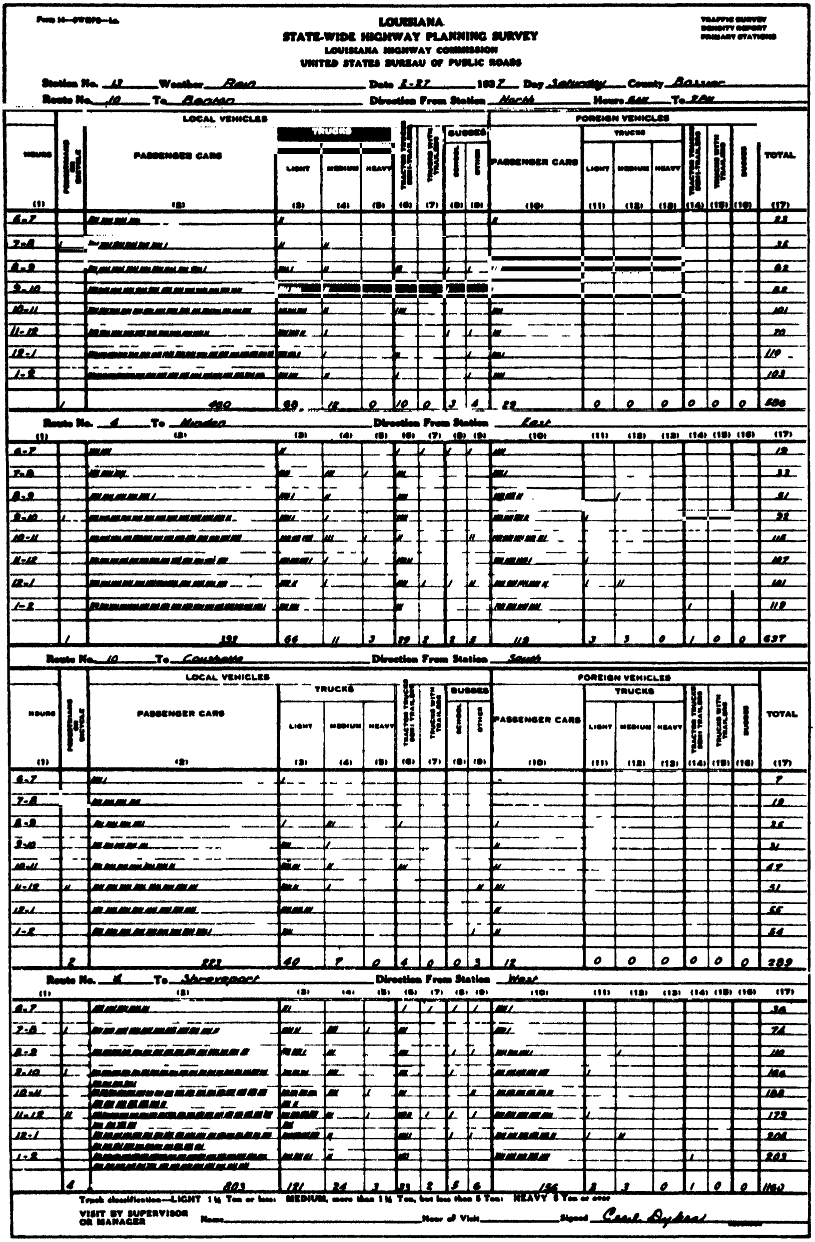
REMARKS______

WEATHER______

OBSERVERS

FIG. 143. Simple form for recording traffic density; when used at intersections one form may be used for each approach; foreign traffic may be recorded on separate forms or by means of colored pencils or a different symbol.

commodities hauled, trip and annual mileages, and other data are recorded on interview forms and on forms distributed to motorists for completion and mailing.



Form 14—SWHPS—La.

LOUISIANA
STATE-WIDE HIGHWAY PLANNING SURVEY
LOUISIANA HIGHWAY COMMISSION
UNITED STATES BUREAU OF PUBLIC ROADS

TRAFFIC SURVEY
DENSITY REPORT
PRIMARY STATIONS

Station No. 13 Weather Rain Date 2-27 1937 Day Saturday County Bossier

Route No. 10 To Benton Direction From Station North Hours 6AM To 2PM

	LOCAL VEHICLES									FOREIGN VEHICLES						
			TRUCKS					BUSSES			TRUCKS					
HOURS (1)	PEDESTRIANS OR BICYCLE	PASSENGER CARS (2)	LIGHT (3)	MEDIUM (4)	HEAVY (5)	TRACTOR TRUCKS SEMI-TRAILERS (6)	TRUCKS WITH TRAILERS (7)	SCHOOL (8)	OTHER (9)	PASSENGER CARS (10)	LIGHT (11)	MEDIUM (12)	HEAVY (13)	TRACTOR TRUCKS SEMI-TRAILERS (14)	TRUCKS WITH TRAILERS (15)	BUSSES (16)	TOTAL (17)
6-7																	23
7-8																	36
8-9																	62
9-10																	82
10-11																	101
11-12																	70
12-1																	119
1-2																	103
	1	480	68	12	0	10	0	3	4	29	0	0	0	0	0	0	596

Route No. 4 To Minden Direction From Station East

(1)		(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
6-7																	19
7-8																	33
8-9																	51
9-10																	92
10-11																	116
11-12																	107
12-1																	101
1-2																	118
	1	392	66	11	3	29	2	2	8	118	3	3	0	1	0	0	637

Route No. 10 To Coushatta Direction From Station South

	LOCAL VEHICLES									FOREIGN VEHICLES						
			TRUCKS					BUSSES			TRUCKS					
HOURS (1)	PEDESTRIANS OR BICYCLE	PASSENGER CARS (2)	LIGHT (3)	MEDIUM (4)	HEAVY (5)	TRACTOR TRUCKS SEMI-TRAILERS (6)	TRUCKS WITH TRAILERS (7)	SCHOOL (8)	OTHER (9)	PASSENGER CARS (10)	LIGHT (11)	MEDIUM (12)	HEAVY (13)	TRACTOR TRUCKS SEMI-TRAILERS (14)	TRUCKS WITH TRAILERS (15)	BUSSES (16)	TOTAL (17)
6-7																	7
7-8																	19
8-9																	26
9-10																	31
10-11																	47
11-12																	51
12-1																	56
1-2																	54
	2	223	40	7	0	4	0	0	3	12	0	0	0	0	0	0	289

Route No. 4 To Shreveport Direction From Station West

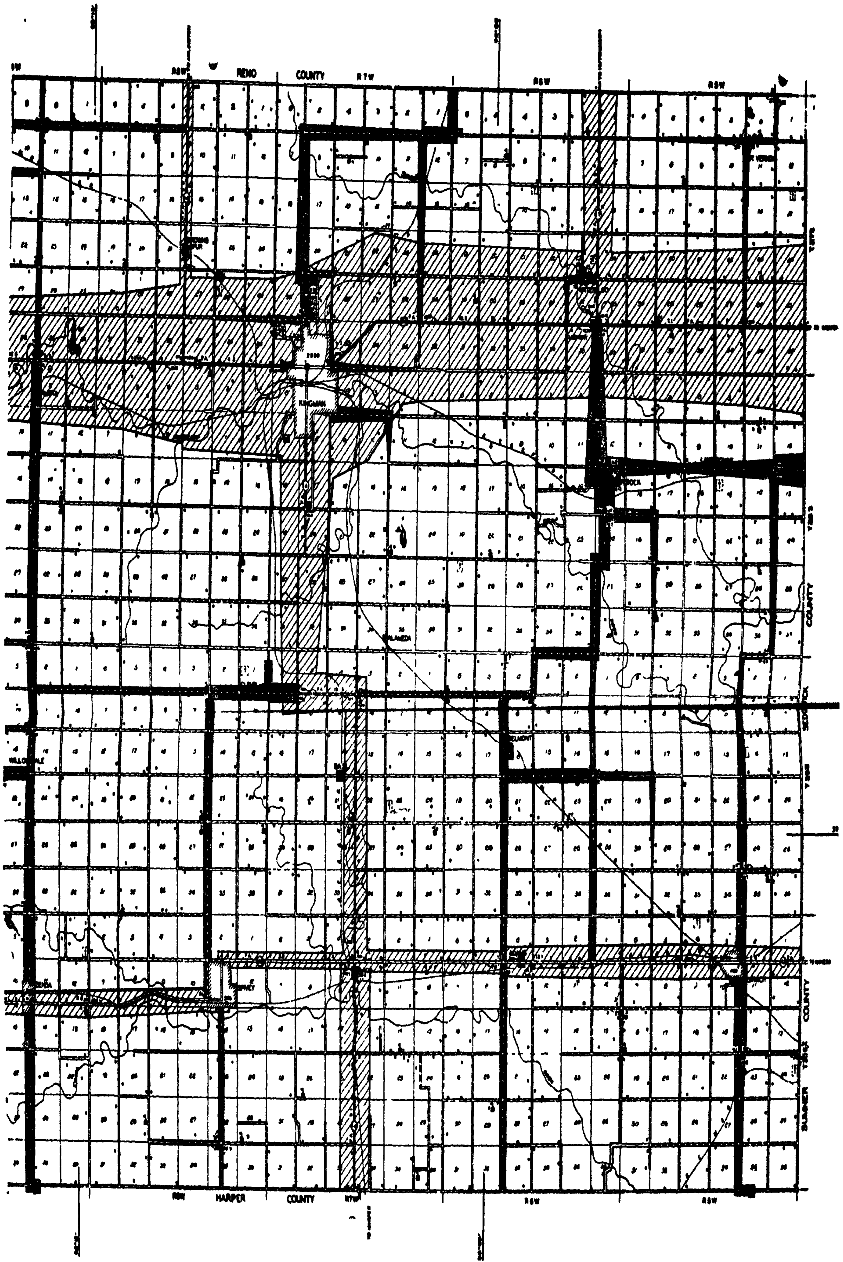
(1)		(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
6-7																	36
7-8																	74
8-9																	140
9-10																	146
10-11																	188
11-12																	179
12-1																	208
1-2																	203
	4	803	121	24	3	33	2	5	6	156	2	3	0	1	0	0	1163

Truck classification—LIGHT 1½ Ton or less; MEDIUM, more than 1½ Ton, but less than 5 Ton; HEAVY 5 Ton or over

VISIT BY SUPERVISOR OR MANAGER Name________ Hour of Visit________ Signed Carl Dykes

Courtesy of Louisiana Highway Commission and U. S. Public Roads Administration.

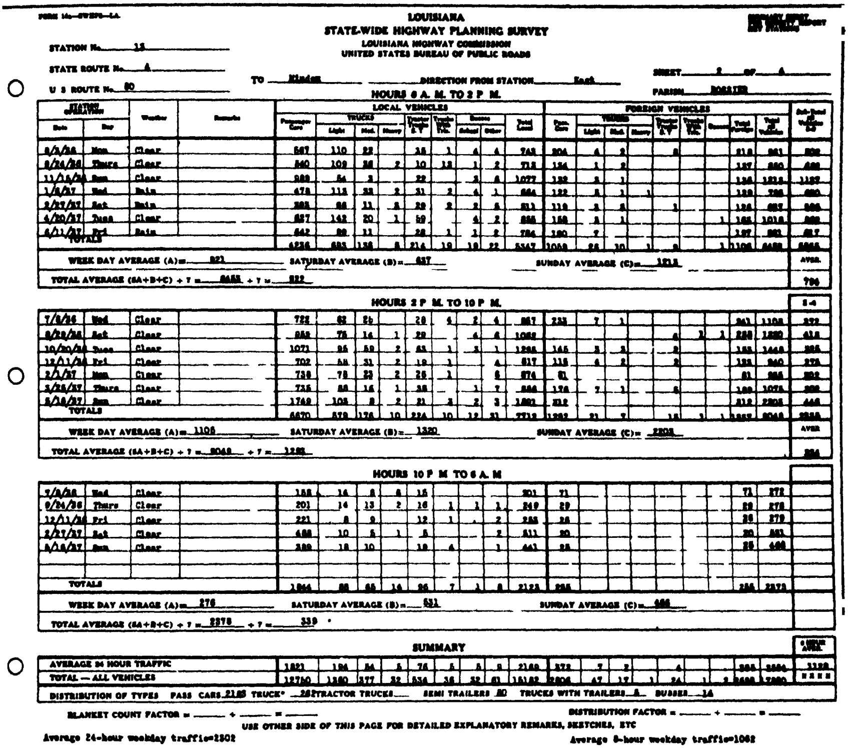
FIG. 144. Form used for recording traffic density on a state highway system.



Courtesy of Kansas State Highway Dept. and U. S. Public Roads Administratio

FIG. 145. Traffic flow diagram for a rural area. The widths of the bands indicate traffic density. The shaded bands represent local highways, and the cross-hatched bands represent state and federal highways.

Automatic Traffic Recorders. Automatic traffic-counting and recording machines which are actuated by the interception, by vehicles, of two light beams directed across the roadway upon photoelectric cells are used to determine the total traffic flow. These machines record the cumulative traffic at regular periods (usually 1-hour intervals) but, of course, do not classify the vehicles. They are particularly useful for determining the accuracy of short-period counts and for establishing the hourly, monthly, and other variations in the total traffic pattern. Thus far these machines have been installed only at locations where electrical power lines are available; when portable machines are developed, they will serve as a very useful aid in determining traffic density.



LOUISIANA
STATE-WIDE HIGHWAY PLANNING SURVEY
LOUISIANA HIGHWAY COMMISSION
UNITED STATES BUREAU OF PUBLIC ROADS

U S ROUTE No. 80

TO Minden DIRECTION FROM STATION East

PARISH BOSSIER

HOURS 6 A. M. TO 2 P. M.

LOCAL VEHICLES

FOREIGN VEHICLES

HOURS 2 P. M. TO 10 P. M.

HOURS 10 P. M. TO 6 A. M.

TOTAL AVERAGE (5A+B+C) ÷ 7 = 2378 ÷ 7 = 339

SUMMARY

AVERAGE 24 HOUR TRAFFIC

TOTAL — ALL VEHICLES

BLANKET COUNT FACTOR = ÷ =

DISTRIBUTION FACTOR = ÷ =

USE OTHER SIDE OF THIS PAGE FOR DETAILED EXPLANATORY REMARKS, SKETCHES, ETC

Average 24-hour weekday traffic=2302

Average 8-hour weekday traffic=1062

Courtesy of Louisiana Highway Commission and U S Public Roads Administration.

FIG 146. Form used for summarizing traffic density

Planning Traffic Surveys. The necessity of careful planning of traffic surveys is obvious. Complexities increase as the area to be covered increases, and efficient and economical procedure must be developed well in advance of the actual observations. Highway organizations, generally, have not done enough of this work and have been deterred largely by lack of funds and because the public, especially in many small local governmental areas, has been slow to appreciate the value of the information secured. Much valuable information can be obtained from a small organization. Frequently it is possible to arrange for maintenance employees to assist at various times.

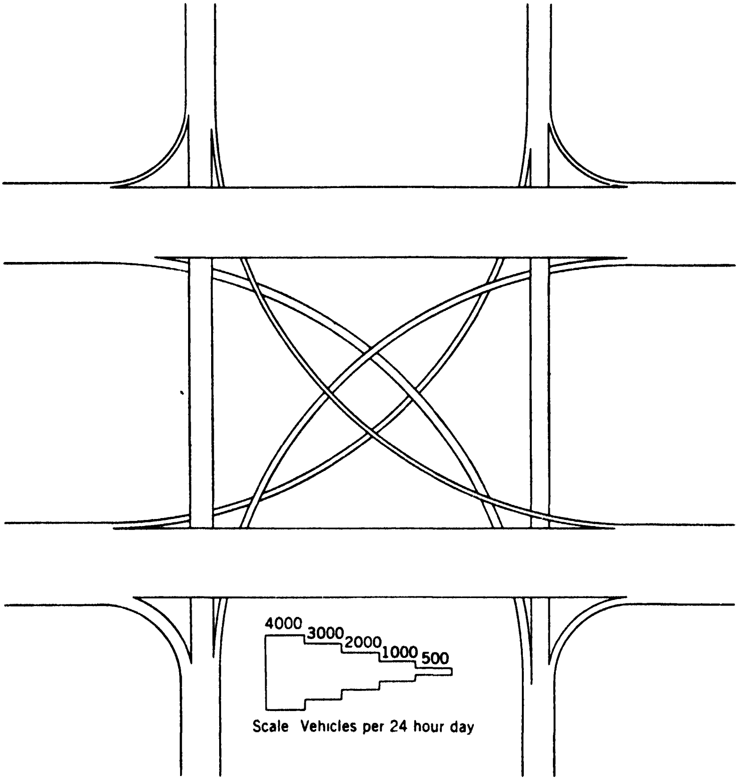


FIG. 147. Traffic flow diagram for a city street intersection.

Compilation of Data. The data obtained from a traffic survey are compiled by means of various summaries and visual charts. Figure 146 shows a form for summarizing the information secured on the form shown in Fig. 144.

Traffic flow diagrams show, visually, the density of traffic on a highway or street system. Figure 145 shows a traffic flow diagram for a rural area in which traffic volumes for individual highways are plotted as bands whose widths are proportional to the amount of traffic. Figure 147 shows a traffic flow diagram for a street intersection.

Traffic distribution diagrams are useful in studying average and maximum traffic volumes at various times and for various periods of time. These diagrams are frequently called *traffic patterns* and may illustrate numerous traffic conditions. They are particularly useful in the design of highways, in the selection of type of wearing surface, in planning maintenance operations, and in the control of traffic. Figures 26 and 27 (Chapter IV) illustrate hourly and daily traffic patterns.

SELECTION OF TYPE OF WEARING SURFACE

The selection of the particular type of wearing surface or pavement for a specific improvement is an important feature of highway administration; it is also one of the most important and, at times, one of the most difficult functions of officials in charge of public highway and street departments. No rigid formulas or rules can be laid down by which selection may be made, but it is possible to describe the influence of numerous factors and conditions which should receive careful and unprejudiced consideration. These factors and conditions are (1) the volume and nature of the traffic, (2) soil and weather conditions, (3) availability of materials, (4) stage construction, (5) economic considerations, (6) financing conditions, (7) maintenance organization and procedure, (8) noise, (9) riding comfort, (10) appearance, (11) dust, and (12) preference of adjacent property owners.

Volume and Nature of Traffic. The nature and volume of traffic on a highway or street are important considerations in selecting the type of wearing surface, but it is very difficult, if not impossible, to formulate precise rules for determining surfacing types because of the great variation in soil conditions, availability of materials, and other conditions. The proportion of heavy commercial vehicles during all seasons of the year is an important factor in the selection of type and, in many cases, may have a much greater influence than a very large number of ordinary passenger vehicles. Through roads, subjected to large volumes of traffic and to moderate or large volumes of heavy commercial vehicles during all seasons of the year, require the heavier and costlier types of wearing courses such as concrete, high-type bituminous pavements, and block pavements; these conditions exist on most primary state highways and on many secondary state highways and primary county highways. The low and intermediate types of wearing courses are adaptable to very light traffic on earth, untreated gravel and sand-clay, and traffic-bound macadam roads, and to moderate to heavy traffic on stabilized soil roads, surface-treated roads, and low-cost bituminous roads on good subgrades. For example, many stabilized gravel roads, low-cost bituminous roads, and cement-bound macadam roads carry up to 500 to 2000 vehicles per day; even some oiled earth roads, during favorable seasons of the year, carry from 300 to 1000 vehicles per day. Careful economic studies may be necessary in many cases to ascertain the relative economy of various surface types.

Soil and Weather Conditions. Soil and weather conditions naturally play a very important part in the selection of the type of wearing surface, especially under some conditions relating to availability of materials. The successful use of stabilized gravel roads depends upon the availability of suitable deposits of gravel and clay binder. The macadam types naturally are constructed where stone is available. Lack of nearby sources of good gravel and stone frequently results in efforts to stabilize the natural soils for surfaces on the lesser traveled roads and in the construction of pavements of imported materials on heavily traveled roads.

Availability of Materials. The lack of nearby deposits of suitable mineral aggregates often eliminates some types of wearing surfaces; the lack of gravel, for example, usually concentrates attention on the improvement of soil roads. Similarly, the lack of suitable sands within reasonable hauling distances often eliminates sheet asphalt and asphaltic concrete. The influence of local materials on the selection of type of wearing surface is illustrated by the use of bituminous macadam in some of the New England States, of gravel in the Great Lakes region, and of rock asphalt in many areas; by the oiling of earth roads in Illinois, the development of sand-clay roads in Georgia, and the use of limerock in Florida, and of shells in some areas along the Gulf Coast.

Stage Construction. The stage construction of highways and streets may be defined as the progressive improvement of a highway or street in several steps or stages, each stage of the improvement being made as traffic conditions change and require more adequate facilities. Stage construction often is considered to apply only to the improvement of low-cost highways, but as a matter of fact all highway improvement necessarily is stage construction.

The adoption of plans for stage construction fixes the selection of type of wearing surface in a general way. Stage construction should be planned so that a change from one type of wearing surface to another enables the maximum advantage to be obtained from prior construction. The principal features of planning for stage construction are:

1. Determination of the importance of the highway and an estimate of the probable changes in traffic which may be expected.
2. Satisfactory location, including plans for future changes.
3. Design and construction of suitable grades for future as well as present traffic conditions. Provisions for future widening may be an important phase of this feature.
4. Obtaining complete information in regard to available surfacing materials and soil conditions.
5. Complete plans for the drainage system, including provision for future development.
6. Plans for progressive changes in the roadway surface, in regard both to character and capacity.

The object of stage construction is to provide, efficiently and economically, for adequate traffic service throughout the development of a highway. Several examples illustrating various plans for stage construction will be described.

Examples of Stage Construction. It is important to keep in mind that all roads will not go through all the stages of development from an unimproved condition to a fully improved thoroughfare for extremely heavy traffic.

CASE I. State or county highway which, upon construction, will be subjected to heavy traffic and on which large future increases in traffic may be expected.

Stage 1: Construction of grade, drainage system and two-lane pavement.
Stage 2: Construction of two additional lanes of pavement.
Stage 3: Resurfacing or reconstruction.

Under these conditions it is desirable to construct the first two lanes of pavement off-center in order to provide for a center parkway or dividing strip.

CASE II. State or county highway which will have light to moderate traffic, changing gradually to heavy traffic.

Stage 1: Construction of grade, drainage system, and gravel surface by the featheredge method in several successive layers.
Stage 2: Construction of stabilized gravel surface. This procedure may follow soon after Stage 1.
Stage 3: Construction of two-lane pavement.
Stage 4: Construction of two additional lanes of pavements.
Stage 5: Resurfacing or reconstruction.

CASE III. State or county highway subjected to gradual increases in traffic but with little probability of large proportions of heavy truck traffic.

Stage 1: Construction of grade, drainage system, and single-lane gravel surface.
Stage 2: Widening of gravel surface to two-lane width (at least 12 in. wider than proposed surface comprising Stage 3).
Stage 3: Strengthening surface by addition of gravel, followed by bituminous surface treatment.
Stage 4: Construction of a 2½- or 3-in. layer of bituminous road-mix or re-tread.
Stage 5: Replacements as needed.

CASE IV. County highway to be used entirely as a land-service highway.

Stage 1: Construction of grade and drainage system; maintained as an earth road.
Stage 2: Construction of gravel surface.
Stage 3: Construction of oil-gravel road-mix wearing course.
Stage 4: Replacements and reconstruction as needed.

Many more cases might be cited to fit many local conditions.

The design and arrangement of many drainage and other structures must be planned carefully to provide for future extensions and additions.

Economic Considerations. Economic considerations involved in the selection of type of wearing surface are the intial cost of the improvement, the annual highway cost, and the cost of transportation over the highway.

The wearing surface or pavement usually represents a very large proportion of the initial cost of construction of a highway project, exceptions being low-type highways and projects involving very large amounts of earthwork, unusually extensive soil stabilization, costly drainage structures, and grade separations. Therefore, the economic selection of the type of wearing surface is very important. It is difficult to generalize in regard to highway construction costs, but the values given in Table 36 show the approximate range of cost of various surfaces.

The annual cost of a highway, so far as road costs are concerned, was stated in Chapter XVIII to consist of (1) interest on the investment, (2) the yearly cost of routine maintenance, (3) the yearly cost of periodic maintenance, and (4) the yearly cost of administration. The cost of routine and periodic maintenance varies greatly for the various types of wearing surfaces and pavements, and experience in a given area should be taken as the basis of comparisons of cost of maintenance. It is somewhat questionable whether administrative costs need be taken into consideration in comparisons of costs for selection of type because it is difficult to assign administrative costs accurately to the various parts of a highway system; some administrative costs are directly proportional to maintenance expenditures, whereas others are more properly considered proportional to road mileage. Interest should be taken into consideration in cost comparisons In general, the type of wearing surface selected for a highway or street should be that type which will have the lowest annual highway cost. This principle does not always hold true, however, because some low-cost roads may show annual costs approximately equivalent to those of high-type pavements when interest on the initial cost of the latter is taken into consideration, and yet would be unsatisfactory on account of interference of maintenance operations with traffic and increased vehicle operating costs.

TABLE 36

AVERAGE COSTS OF CONSTRUCTION OF VARIOUS TYPES OF ROAD SURFACES AND PAVEMENTS

(Grading, drainage structures, and miscellaneous structures are not included.)

Type	Average Cost of Construction		
	Per Sq Yd	Per Mile	
		18 Ft Wide	20 Ft Wide
Gravel	$0.20 to $0.80	$2,112 to $8,448	$2,347 to $9,386
Waterbound macadam	0.65 to 1.40	6,864 to 14,784	7,626 to 16,426
Bituminous surface treatment	0.07 to 0.20	739 to 2,112	821 to 2,347
Low-cost bituminous surface	0.20 to 0.50	2,112 to 5,280	2,347 to 5,867
Bituminous macadam	1.25 to 2.00	13,200 to 21,120	14,666 to 23,466
Asphaltic concrete on concrete base	2.25 to 3.50	23,760 to 36,960	26,399 to 41,066
Sheet asphalt on concrete base	2.50 to 4.00	26,400 to 42,240	29,333 to 46,932
Concrete, 9-7-9 in	1.50 to 2.75	15,840 to 29,040	17,600 to 32,266
Brick on concrete	3.00 to 5.00	31,680 to 52,800	35,200 to 58,665
Stone block on concrete	4.50 to 9.00	47,520 to 95,040	52,800 to 105,600
Wood block on concrete	4.50 to 9.00	47,520 to 95,040	52,800 to 105,600

The effect of the type of wearing surface on the cost of vehicle operation was discussed in Chapter XVIII. Differences in cost are most apparent in comparisons of low types of surfaces, such as earth and gravel, with the higher types. It is questionable that differences between surfaces in the same general class are sufficient to govern the selection of type.

Financing Conditions. It is obvious that financing conditions necessarily must affect the selection of type of wearing surface. When all highway revenues are fixed, as when motor-vehicle revenues, for example, are allocated to both construction and maintenance and no other funds can be provided, the undertaking of construction projects often is a difficult problem and the selection of the proper type of wearing surface for a specific project may not follow the soundest economic principles.

Most street paving or surfacing is financed in whole or in part by assessments on the abutting property, and property owners naturally desire to participate in the selection of the type of surfacing or pavement. The author has frequently heard the statement that property owners should have no part in the selection of type of surface, but he cannot fully agree with this principle; in the case of a residential street used almost exclusively as an entrance to property, and for which the abutting property owners must pay the entire cost or a large share of the cost, it seems only reasonable that the wishes of the property owners should receive careful consideration. Many laws attempt to control the selection of type by means of hearings conducted according to fixed procedures. The present circumstances surrounding such work are improved over former conditions when the early steps in the procedure to surface a pavement or street were the signal for promoters and salesmen to invade the community in droves and attempt by every means to have their particular types adopted. Sane procedure, under such conditions, was practically impossible.

Maintenance Organization and Procedure. The selection of the type of wearing surface or pavement for a particular project is intimately related to the type of procedure and organization of maintenance. From a strictly maintenance viewpoint it is desirable to have a few different types of surfaces to maintain rather than a large number of types; maintenance procedure is simplified, standard operations are more easily developed, and costs are lowered generally.

Noise. The element of noise in connection with the selection of type of wearing surface is of much less importance than formerly because of the almost total exclusion of horse-drawn, steel-tired vehicles.

Riding Comfort. The development of modern pneumatic tires and shock-absorbing devices on motor vehicles and of procedures for obtaining smooth surfaces on almost any type of wearing course eliminates the necessity of giving much consideration to the feature of riding comfort.

Dust. Dust is a serious inconvenience on untreated earth, gravel, shell, and broken-stone roads and has resulted in the restriction of these surfaces to light-traffic roads and streets.

HIGHWAY FINANCING

The principal problems of highway finance are:

1. The economic improvement of highways.
2. The equitable distribution of the cost of highway improvement and maintenance.
3. Proper limitation of the use of credit to provide immediate funds for highway improvement.

Economic Improvement of Highways

Economy in the improvement of highways has been defined by the American Association of State Highway Officials in the statement that "no road should be improved by expenditure of funds in excess of its earning capacity." The earning capacity of a highway depends upon the uses of the highway and the relative value of each form of usage. The value of each form of highway usage in turn depends upon the benefits obtained.

Benefits of Improved Highways. The recipients of benefits from improved highways may be classified into the following groups:

1. The traffic which uses the highway.
2. The owners of adjacent property.
3. The general public.

Benefits to Traffic. The benefits of improved highways to the people who use them comprise:

1. Decreased cost of transportation.
2. Travel comfort.
3. Saving of time.
4. Recreational and educational benefits.
5. Commercial benefits.

Benefits to Adjacent Property. In almost all instances, abutting or adjacent property receives benefits from highway improvement. These benefits consist of:

1. Access to property.
2. Increased value of the adjacent property.

3. Extension of outlets for farm produce or commodities.
4. More efficient postal service.
5. Improvement of sanitation.
6. Locations for public utilities, such as electricity, telephone service, water and gas mains, sewers, and drainage.
7. Increased effectiveness of fire and police protection.
8. The value of streets as fire barriers.
9. Removal of refuse, such as garbage, ashes, and other debris.
10. Means of access of light and air to buildings (in cities).
11. Value to pedestrian traffic.
12. Parking space for vehicles.

Benefits to the General Public. The benefits of highway improvement to the general public consist of:

1. Development of the territory.
2. Decreased costs of commodities.
3. More effective fire and police protection.
4. Educational advantages.
5. Sociological advantages resulting from the convenience of transportation.
6. Beautification of urban or rural property.
7. Improved national defense.

Evaluation of Benefits. Some of the benefits of improved highways are capable of reasonably accurate measurement, but others are more difficult to evaluate because of their intangible nature. It is quite certain that not all benefits accrue in direct proportion to highway costs. Reference to the cost of the various elements of the highway structure throws some light on the evaluation of the benefits or uses of a highway. The highway structure has already been described in Chapter I as being composed of: (1) the subgrade, (2) the roadway or traffic-way, (3) the shoulders, (4) the roadside, (5) the drainage system, and (6) accessory structures. The most costly features of the highway structure are the roadway or wearing surface and such structures as bridges and grade separations. It is at once evident that, on rural roads, at least, the most expensive features of highway improvement are those which are occasioned almost solely by the *amount* and character of the traffic.

Evaluation of Traffic Benefits. The justifiable expenditure for highway improvement will depend in a large measure upon the contribution that traffic can afford to pay and upon the volume of the traffic. The contributions of traffic to the cost of highways usually consist of a license fee on the vehicle itself and a tax upon the fuel that is consumed. Let us assume that the average highway vehicle uses the highways to the amount of 10,000 miles annually, travels 14 miles on each gallon of gasoline, and pays a license fee of $15 and a fuel tax of $0.05 per gallon. The annual payment for the privilege of using the highways then is as follows:

License fee	$15.00
Gasoline tax, 714 gal. at 0.05	35.70
Total payment	$50.70
Payment per mile	$0.005

The annual traffic income for various rates of traffic then will be as follows:

Traffic Volume, vehicles per day	Traffic Income, per mile, per year
50	$91.25
100	182.50
250	456.25
500	912.50
750	1368.75
1000	1825.00
1500	2737.50
2000	3650.00
5000	9125.00

If a saving of ½ to ¾ cent per mile in the cost of operation of a motor vehicle accrues when a road is properly improved and adequately maintained, it is evident that this contribution on the part of a motor vehicle is not excessive. Carefully conducted investigations have shown that savings of such amounts are generally realized with reference to fuel consumption alone.

The relation of traffic income to highway costs is difficult to determine because of the many variable factors involved. It is apparent, however, that traffic income on low-traffic roads will be insufficient to pay all costs of highway improvement and maintenance. It also is apparent that traffic income on some heavily traveled highways will be more than sufficient to pay all improvement and maintenance costs.

Evaluation of Benefits to Adjacent Property. The accurate evaluation of the benefits of highway improvement and maintenance to the adjacent property is a difficult problem because of the intangible nature of the benefits and because of the great variation in the value of the benefits. Some authorities place emphasis on the theory that all benefits to adjacent property are reflected in an increase in the market value of the land and advocate contributions based on such increase in value. It should be apparent that the benefit to adjacent property is not directly proportional to the extent of the cost of improvement and maintenance made necessary by the amount and character of the traffic and that, under some conditions, highway improvements may actually be detrimental to land values. Thus the benefits to adjacent property reach a *maximum value* at some stage of highway improvement. That some *minimum value* also exists is also self-evident; this minimum value probably represents the value accruing from the improvement and maintenance of a highway as a means of access to the adjacent property.

Evaluation of Benefits to the General Public. The evaluation of general benefits resulting from highway improvement and maintenance is even more difficult than for the other benefits. A sound conception of limitations on total public expenditures will have an important bearing on a satisfactory solution of this important problem.

The expenditure of public funds may be classified as follows:

1. Government.
2. Public defense and safety.
3. Education.
4. Public health and general welfare.
5. Social welfare.
6. Public works.

Funds for these purposes are expended by several units of government including the federal government, the state government, and various local units. These funds are obtained from public taxation in some form or another. Constitutional and statutory limitations on taxation are necessary to prevent abuse of the taxing power, either by public officials or *by the public, itself.*

It is apparent that a limitation on the *total amount* of money raised for public purposes is necessary as well as a limitation on the amount for single purposes. Highway expenditures, necessarily, must be controlled, and this control should be a feature of a comprehensive plan of taxation, based on needs and ability to pay.

ASSESSMENT OF HIGHWAY BENEFITS

If highway costs were assessed in strict accordance with the benefits received, the expenditures on each highway or on each highway system should be assessed against all three types of beneficiaries, namely:

1. The road vehicles or road users.
2. The adjacent property.
3. The general public, in the form of
    (*a*) The local community or communities.
    (*b*) The state.
    (*c*) The federal government.

Practically, this procedure would be cumbersome, slow, and costly. The tendency in the United States has been to allocate certain ways or methods of taxation to various divisions of the government structure.

Methods of Assessing Highway Costs. The various methods of assessing highway costs include the following:

Assessments on road vehicles:
   1. Vehicle license fees.
   2. Fuel taxes.
   3. Sales taxes on the vehicle and on parts and supplies.

Assessments on adjacent property:
   1. Special assessments levied on the property.

Assessments on the general public:
   1. General property taxes.
   2. Sales taxes.
   3. Income and other taxes.

Motor Vehicle License Fees. The license fee for motor vehicles was originally instituted for the purpose of controlling the operation of vehicles. When it first became evident that motor vehicles should contribute toward the cost of highway improvement, the license fee provided a convenient means of collecting such a contribution. All the states, and some cities, impose license fees on the basis of weight, horsepower, cost, type of tires, and nature of use. City license fees are generally known as "wheel taxes." In some states the vehicle license fee is in lieu of property taxes; this situation sometimes makes the financing of city street improvements and maintenance difficult.

Fuel Taxes. The fuel tax is one of the most equitable forms of taxation for highway purposes because it adjusts the contribution of the vehicle in proportion to use. An objection to this tax because "the worse the road, the more the tax" is more apparent than real in the present stage of highway development, because the best roads now are usually those which receive the most revenue from vehicles, whereas the poorer surfaces are those to which vehicles should contribute the least. Fuel taxes are imposed by the states and by the federal government, and, occasionally, by local units of government.

General Property Taxes. General property taxes have been the favorite means of assessment of benefits to the general public, and, in many instances, the taxing power has been abused. Some states have placed restrictions on total property taxes to curb this abuse.

Special Property Assessments. The most practicable method of assessing adjacent property for benefits resulting from highway improvement is by means of a special property assessment. This method of property taxation, like general property taxation, however, has been greatly abused. Special assessments against adjacent property are commonly devoted by municipalities to financing all, or a part of, street improvements and are not so widely used for rural road improvements; in some instances, the abutting property, only, is subject to a special assessment for the improvements; in others, the property on each side of the road or street is divided into zones of certain widths with decreasing assessments according to the distance of the property from the improvement.

Sales and Income Taxes. Sales taxes and income taxes are gradually replacing general property taxes as a source of revenue for the operation of state governments. It is questionable whether a large portion of highway costs should be obtained from these taxes.

ALLOCATION OF HIGHWAY REVENUES

The impracticability of assessing all three types of beneficiaries of highway improvement for each road or street or for each type of highway system is apparent. Financing methods vary in every state, and they have been modified and altered in almost every state from time to time. A logical solution of the distribution of highway revenue will depend to a large extent upon the use of the various highways. The proportion of traffic on the various portions of our rural highways is not constant, of course, but is greatest on the state highway systems, less on main county highways, and least on purely land-service roads. The extent of the rural highways of the United States, and the proportion of traffic which each system bears, are approximately as follows:

	Miles	Per Cent of Total Traffic Carried	Per Cent of Use	
			City Vehicles	Rural Vehicles
State highway systems....	300,000	65	75	25
Main local highways.....	600,000	25	55	45
Local land service roads. .	2,200,000	10	35	65

It is evident that the motor vehicle should contribute the largest portion of the cost of the state highway systems and of main local highways. It also should be evident that the other beneficiaries of highway improvements should contribute a large proportion of the cost of local and land-service roads and, perhaps, some of the cost of main local highways.

Financing State and Interstate Highways. The major use of state and interstate highways is by through-traffic; motor-vehicle revenues, therefore, should furnish a very large proportion of the cost of improvement and maintenance. Contributions by the federal government are justified on these routes because they carry large proportions of interstate traffic.

Financing Local Rural Highways. The financing of local rural highways in some areas is a difficult problem during the development stage of highway improvement, and property taxes have been used to a large extent. The desire of many communities for types of improvements which are more costly than necessary has resulted in larger expenditures than economic conditions warrant. Vehicles should contribute a large share of the cost of improving and maintaining intercity and intercommunity routes, but it is questionable whether they should be called upon to contribute the major portion of the cost of purely land-service roads. Outstanding indebtedness for the improvement of many county road systems, together with uneconomic design, has delayed desirable improvements on land-service roads.

The allocation of motor-vehicle license fees to local road costs, and of fuel tax receipts to state highway expenditures, in the more populous and wealthy states, is a practicable method of distributing motor-vehicle contributions, but in many cases will provide insufficient funds to improve all classes of roads rapidly.

Financing City and Village Streets. The financing of city and village streets usually has been accomplished by means of local taxes in the form of special property assessments and general property taxes. The extent of participation by urban municipalities in the income from motor-vehicle revenues has been a much-debated question in many states. The problem becomes acute when the license fee is in lieu of property tax on vehicles. Cities and villages certainly should receive some contribution from motor vehicles for the improvement and upkeep of general traffic thoroughfares. The allocation of a portion of motor-vehicle revenues, by means of a personal property tax, a municipal license fee, or a share in the total license fee or fuel tax paid by municipally owned vehicles, partly solves the problem. It also is entirely logical for state and county highway departments to contribute to the cost of state and county routes in municipalities, and this practice is followed in numerous states.

BOND ISSUES FOR HIGHWAY IMPROVEMENT

The payment of the cost of improvements which cannot be conveniently financed out of current revenues may be deferred by bond issues; many highway and street improvements have been financed in this way.

Types of Bonds. Bonds may be classified into three principal types: sinking-fund bonds, annuity bonds, and serial bonds. Sinking-fund bonds are issued for a single date of maturity and are retired by means of a sinking fund accumulated by uniform annual tax installments which are assumed to be invested in interest-bearing securities. Annuity bonds have varying maturity dates and are retired by constant annual payments embracing both principal and interest. Serial bonds have varying maturity dates and are retired by constant annual payments of principal plus constantly decreasing interest payments. Modified serial bonds are widely used and have deferred or varying payments on the principal.

Sinking-fund bonds have several inherent disadvantages. The sinking fund usually is subject to variations in income which cannot be predicted, and its presence is a constant temptation to officials to use it for other than the intended purpose. The use of sinking funds for purchase of new bonds issued by the same governmental unit introduces complications in governmental financing. In addition, pressure of other needs or periodic changes in economic conditions may tempt officials to defer assessment of the annual tax installments.

Annuity bonds possess the single advantage of constant annual tax installments, whereas the annual tax installments of true serial bonds are constantly decreasing. A comparison of the cost of a loan retired by means of annuity and serial bonds is given in Table 37.

Maturity of Bond Issues. Theoretically, a debt should be discharged by the time the item for which the debt was incurred has reached the end of its useful life. A highway is made up of several elements, each of which may have varying lengths of life; the cost of grading, for example, may be a practically permanent investment, whereas the wearing surface may remain intact for only a comparatively short time. When bonds are of the serial type and highways are improved with high types of pavements, maturities of 20 to 25 years may be feasible. Bonds for bridges of a lasting type may run for longer periods.

Security for the Payment of Bonds. When bonds were first issued to assist in the financing of public improvements, their payment was secured by pledging the assessment of taxes against real property, and the taxes thus assessed became liens against the land and improvements involved. Many bond issues are still secured by the payment of property taxes, but the payment of many recent bond issues has been pledged from revenues other than property taxes. Thus provision for payment may be made from revenues from public-utility services and from income derived from motor-vehicle license fees and fuel taxes.

Theory of Bond Issues and Limitations on the Use of Credit for Highway and Street Improvements. The theory underlying the issuance of bonds for highway purposes is twofold: first, the economic return on the improvement, if provided immediately, is more than sufficient to pay the interest on the bonds; and second, future generations will share in the benefits of the improvement and should therefore bear a portion of the cost of the improvement.

The extensive use of credit for highway purposes is a dangerous procedure because the future cannot be accurately predicted. Many public organizations which had issued bonds for public improvements to be paid from property taxes found themselves unable to meet maturity obligations during the financial depression which began in 1929, and many extensive refinancing programs thus became necessary.

ADMINISTRATIVE CONTROL AND ORGANIZATION OF HIGHWAY DEPARTMENTS

The administrative control of highway systems in the United States varies greatly in the different states.

The administrative control of state highway systems in the United States generally is vested in a state highway commission, a state department of public works, or a single administrator. Generally, the administrative officers are appointed by the governor of the state, but some are elected, and some are ex officio members.

Unfortunately, appointments of administrative officers of state highway departments too often are political in character and are not made on the basis of fitness for the office. The ideal form of organization would be patterned after large business organizations and would consist of a commission composed of three to five men appointed for their outstanding ability, integrity, and interest in public affairs, who would select a capable executive to have responsible charge of the operation of the organization. A capable engineer with extensive administrative as well as engineering experience generally would be the most suitable type of executive officer, but a good business administrator who

TABLE 37

Schedules of Interest and Principal Payments to Retire a Loan of $100,000 at 4 Per Cent in 20 Years by Means of Annuity Bonds and Serial Bonds

(*Data from U. S. D A. Bul.* **136**)

Year	Annuity Bonds*				Serial Bonds			
	Principal Outstanding at Beginning of Year	Interest at End of Year	Principal Retired at End of Year	Total Yearly Payment	Principal Outstanding at Beginning of Year	Interest at End of Year	Principal Retired at End of Year	Total Yearly Payment
1	$100,000	$4,000	$3,400	$7,400	$100,000	$4,000	$5,000	$9,000
2	96,600	3,864	3,500	7,364	95,000	3,800	5,000	8,800
3	93,100	3,724	3,600	7,324	90,000	3,600	5,000	8,600
4	89,500	3,580	3,800	7,380	85,000	3,400	5,000	8,400
5	85,700	3,428	3,900	7,328	80,000	3,200	5,000	8,200
6	81,800	3,272	4,100	7,372	75,000	3,000	5,000	8,000
7	77,700	3,108	4,200	7,308	70,000	2,800	5,000	7,800
8	73,500	2,940	4,400	7,340	65,000	2,600	5,000	7,600
9	69,100	2,764	4,600	7,364	60,000	2,400	5,000	7,400
10	64,500	2,580	4,800	7,380	55,000	2,200	5,000	7,200
11	59,700	2,388	5,000	7,388	50,000	2,000	5,000	7,000
12	54,700	2,188	5,200	7,388	45,000	1,800	5,000	6,800
13	49,500	1,980	5,400	7,380	40,000	1,600	5,000	6,600
14	44,100	1,764	5,600	7,364	35,000	1,400	5,000	6,400
15	38,500	1,540	5,800	7,340	30,000	1,200	5,000	6,200
16	32,700	1,308	6,000	7,308	25,000	1,000	5,000	6,000
17	26,700	1,068	6,300	7,368	20,000	800	5,000	5,800
18	20,400	816	6,500	7,316	15,000	600	5,000	5,600
19	13,900	556	6,800	7,356	10,000	400	5,000	5,400
20	7,100	284	7,100	7,384	5,000	200	5,000	5,200
		$47,152	$100,000	$147,152		$42,000	$100,000	$142,000

* In denominations of $100.

knows how to cooperate with a technical staff often makes a very satisfactory executive. The commission would then meet at frequent intervals to adopt general policies, approve budgets and expenditures, and exercise general control over the acts of the executive officer. This form of organization possesses the advantage of being able to command the services of outstanding men who could not afford to devote their full time to the work.

The administrative control of county highway systems, like that of state highway systems, varies greatly in the different states. In some states, county highways are directly under the control of a general county board which is elected by the people to administer all county business. This method is generally ineffective and often is no improvement over the township plan of administration. In other states, administration is vested in a superintendent or other official who is appointed by the general governing body of the county. This plan is usually more effective than the first method described but generally is entirely political in character. In still other states, county highways are administered by a special board or commission which is elective or appointive. When men of outstanding ability and character are induced to serve on such a board and leave the details of administration to a capable executive, this plan is most effective. It is a somewhat surprising fact that extremely able men can be induced to serve on commissions of this kind for very nominal compensation. This plan has been widely followed in Michigan and some other states and has been an outstanding success in many counties where men have been appointed whose only interest is in good management of public affairs. It is not popular, of course, with politicians, because, when properly administered, it eliminates political patronage.

The care of township highways usually is entrusted to a highway commissioner who is elected by the people or is appointed by a general township board.

Organization of Highway Departments

The organization of highway departments is developed in accordance with the particular functions which are to be fulfilled and varies according to state laws, type of road system, and the opinions and desires of administrative officials.

Functions of State Highway Departments. The principal duties and responsibilities of state highway departments generally are as follows:

1. *Participation in the selection and establishment of the state highway system.* The state highway departments of some states have very broad powers in the selection of the roads which form the state highway system. In many states the general location and direction of each route are described by legislative enactment, and the details of location are left to the discretion of the department.
2. The construction and maintenance of the state highway system, including federal-aid roads.
3. The distribution of state aid for highways to local units of government.
4. Cooperation with local units of government by furnishing advisory service.
5. The dissemination of information to the public through the medium of reports, meetings, specifications, and by other means.

Numerous other duties, which vary greatly in the different states, are imposed on state highway departments. Some departments collect motor-license fees, others have charge of policing duties, and still others are required to administer certain regulatory functions.

Functions of County Highway Departments. The principal functions of county highway departments are:

1. The selection of the roads which constitute the county road system. This function applies only in those states which also have a town, or township, road system.
2. The construction and maintenance of the county highway system.
3. The establishment of new public highways. Necessarily, the partition of property and other rural developments requires the establishment of new highways from time to time.

Functions of Township Highway Departments. Township highway departments generally are concerned only with roads of purely local importance, and their duties consist of the improvement and maintenance of these roads and the establishment of new roads.

Distribution of Authority and Responsibility. The work of state highway departments is usually arranged into several divisions, approximately as follows:

1. The administrative body or officer.
2. A division of planning.
3. A division of design.
4. A division of construction.
5. A division of maintenance.
6. A division of tests and research.
7. A division of public relations.
8. A division of finance or accounts.
9. A legal division.

Some of the larger county highway departments are similarly organized; in others, a less-specialized form of organization is adequate.

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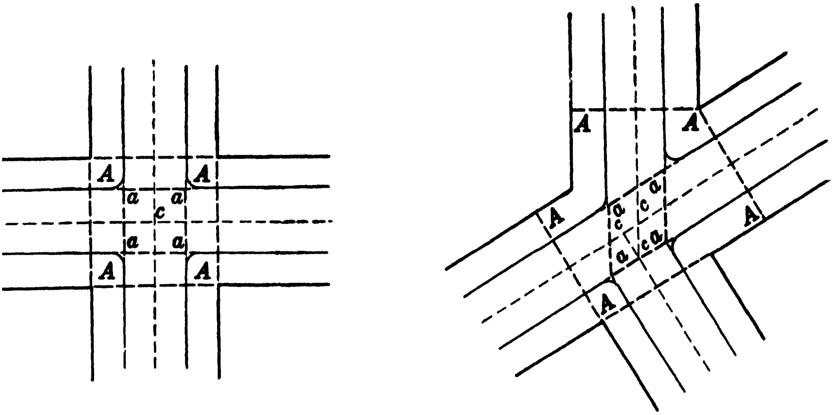
APPENDIX A

DETERMINATION OF STREET INTERSECTION ELEVATIONS

(*Moon's Method*)[1]

1. *Definition of Platforms.* The center line intersection shall be deemed to be the point of intersection of the center lines, except for cases where the center lines do not meet at a common point, when it shall be the area included within the center lines at their intersection (point *c* and area *ccc* in Fig. 1).



(*a*) Right-angled Intersection **(*b*) Acute-angled Intersection**

FIG. 1.

The curb line platform shall be deemed to comprise the area included within the lines connecting the points of intersection of the curb tangents, or in the case of a street terminating at another street it shall comprise the area within the prolongations of the curb lines across the intersection and a line joining the curb tangents (areas *aaaa* in Fig. 1).

The building line platform for rectangular intersections shall be deemed to include the area bounded by the prolongations of the building lines of both streets across the intersection so as to comprise the greatest platform area. In the case of other than right-angled intersections, it shall comprise the area bounded by the respective lines of each street and by lines at right angles or normal to the center lines and passing through acute-angled building line corners, or the corners giving the greatest platform area. If the intersection of the center lines falls without the Building Line Platform, as above described, the said platform shall be increased sufficiently to include the said intersection. When the building line corner is turned with a curve the platforms above defined shall be indicated upon the map unless herein definitely fixed. (The Building Line Platform is indicated in Fig. 1 by the lines *AAAA*.)

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[1] Moon, V. S., "Proposed Method of Interpreting Grade Elevations," *Proc. Municipal Engineers of the City of New York,* 1911, p. 40.

2. *Definitions of Elevations Fixing Grades.* Unless otherwise indicated on the map, the elevations shown at a street intersection shall be deemed to be that fixed for the point of intersection of the center lines of both streets affected, or for the Center Line Intersection.

3. *Treatment of Center Line Intersection.* The Center Line Intersection, when it comprises an appreciable area and unless otherwise shown on the map, shall have a uniform elevation at its boundaries, and in determining the elevations for the other platforms herein described the Center Line Intersection referred to as a basis of calculation shall be deemed to be the nearest point on the center line of each street at the boundary of the said platform.

4. *Treatment of Platform for Streets Having a Light Grade.* If the grade of each of the intersecting streets is 3 per cent or less, as determined by calculating the rate between the established elevations, the elevation of the center lines of each street within the limits of the Curb Line Platform shall be the same as that fixed for the Center Line Intersection. The elevation of the curbs shall be determined as indicated in Paragraph 8. Provided, however, that the difference in the elevation of points on the center lines opposite any building line corner shall not provide a greater transverse sidewalk slope than that fixed as the maximum in Paragraph 7, in which latter event the Building Line Platform shall be used and the grades of that portion of the streets adjoining the said corners shall be flattened between the boundaries of the Building Line Platform and the Center Line Intersection, as provided in Paragraph 5(*a*).

5. *Treatment of Platform for Streets Having a Steep Grade or Meeting at an Acute-angled Intersection.* (*a*) If the grade of any portion or portions of intersecting streets adjoining a building line corner is over 3 per cent, as calculated between the established elevations, or if a further flattening of the platform grade is required to provide proper sidewalk slopes for any part of an intersection described in Paragraph 4, the grades of the said portion or portions of each street shall be reduced between the boundaries of the Building Line Platform and the Center Line Intersection as follows: If the intersecting streets are of the same width, the grade of the street traversing the shorter block length adjoining the intersection shall be reduced one-third and that of the street traversing the longer block shall be reduced two-thirds. In case the streets have different widths, the grade of the wider street shall be reduced one-third and that of the narrower street two-thirds between the above limits. All grades less than 3 per cent which are not herein required to be flattened shall be

applied at the same rate as originally computed between established elevations; provided, that in no case shall the maximum platform and sidewalk slopes fixed in Paragraphs 6 and 7 be exceeded.

Any excess in grade over that allowed in Paragraph 7 shall be removed by further flattening, as follows:

(*b*) *Special flattening of platform grades for extreme cases of steep grades or acute-angled intersections.* If the difference in elevation tentatively fixed for points on the center lines of intersecting streets opposite any building line corner, after applying the minimum and up to maximum transverse sidewalk slope on the higher and lower sides respectively, exceeds the maximum transverse sidewalk grades hereinbefore described, the elevation of each street at the boundary of the Building Line Platform shall be adjusted to remove the excess, the adjustment of each of the said elevations being directly proportional to the grade of each as originally flattened or applied.

For all cases covered by Paragraphs (*a*) and (*b*) the elevations at the intersections of the center line of each of the narrower streets or at the streets traversing the longer blocks, if they are of equal width, with the Curb Line Platform of the intersected streets shall be the same as the elevation of a point on the center line of the intersected street immediately opposite the first-named intersection, except that the elevation at this point shall be abandoned when the grade along the center line between the Curb Line Platform and the Building Line Platform exceeds the grade as originally computed.

The grades of the center line of the wider street or of the street traversing the shorter block, if they are of equal width, shall be uniform between the exterior boundaries of the Building Line Platform and the Center Line Intersection, except that the maximum platform slope hereinafter fixed shall not be exceeded. The grades of the center line of the narrower street or of the street traversing the longer block, if they are of equal width, shall be uniform between the elevations fixed at the exterior boundaries of the Building Line Platform and those fixed at the boundaries of the Curb Line Platform, and also between the latter point and the Center Line Intersection.

6. *Maximum Platform Grades.* The maximum allowable grade along the center line between the Curb Line Platform and the Center Line Intersection shall be at the rate of 4 per cent, unless otherwise indicated on the map.

The grades along the center line between elevations established within the limits of a Building Line Platform shall be uniform, subject only to the flattening provided for in Paragraph 5(*b*).

7. *Transverse Sidewalk Grades.* Whenever practicable, the sidewalk shall slope upwards in a direction at right angles to the curb toward the building line at the rate of 2 per cent.

The elevation of the sidewalk at the building line corner shall be determined by applying this rate to the elevation of the curb, giving the higher building line elevation, at a point immediately opposite the corner, unless the resulting grade on the lower side exceeds 6 per cent, in which case the sidewalk shall be level on the higher side and a greater transverse sidewalk slope up to the maximum shall be used on the lower side.

The maximum transverse sidewalk slope shall be 6 per cent, except in those cases where the street grade as originally computed on any street adjoining a building line corner is more than 6 per cent, when the maximum slope shall be 10 per cent for either street, opposite the said corner. In no case shall the sidewalk at the building line be lower than that of a point immediately opposite it on the curb. If the transverse sidewalk slope at the building line corner is more or less than 2 per cent, it shall be made to agree with this latter rate at a point distant 25 ft from the building line corner.

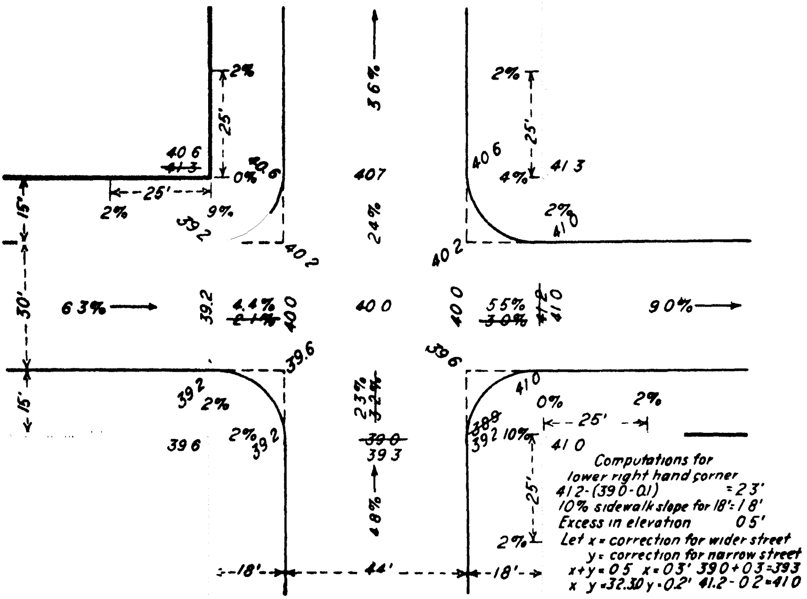

FIG. 2.

8. *Curb Elevations.* The relation between the elevation of the center lines and of the top of the curbs at points immediately opposite it at the boundary of and outwardly from the Building Line Platform shall be as follows: For roadway widths of 24 ft or less the top of the curbs shall be 0.1 ft higher than the center line. For roadway widths ranging from 24 ft up to and including 34 ft the top of the curbs and the center line shall be at the same elevation. For roadway widths ranging from 34 ft up to and including 44 ft the top of the curbs shall be 0.1 ft lower than the center line. For roadway widths ranging from 44 ft up to and including 54 ft the top of the curbs shall be 0.2 ft lower than the center line, and for roadway widths ranging from 54 ft up to and including 64 ft the top of the curbs shall be 0.3 ft lower than the center line.

The elevation of the intersection of the curb tangents shall be determined from a point immediately opposite on the center line of the wider street or the street traversing the shorter block, if they are of equal width, subject, however, to the same correction in elevation between the top of the curbs and the center line as herein provided.

9. *Depth of Gutters.* Whenever practicable a standard depth of gutter of 0.4 ft shall be used.

10. *Curb Grades at Corners.* The tangents in the curbs shall be graded uniformly between the elevations established for them at the boundaries of the Building Line Platform and at the intersection of the curb tangents. The curve formed in the curb joining the tangents shall follow a uniform grade between the elevations of the curb tangents at the points of curve.

11. *Grades between Platforms.* The grades of the center line and of the curbs between the elevations computed at platform intersections, or between a platform and an intermediate established elevation, shall be uniform.

Figure 2 illustrates the design of an intersection according to this method.

APPENDIX B

TABLES FOR DETERMINING VOLUMES OF EARTHWORK

Directions for Using the Tables

The figures at the top and bottom of Table A represent the number of "hundred" cubic yards for the sums of the end areas (in square feet) given in the body of Table A. The figures in the body of Table B represent the balance of the volume (in cubic yards) for "tenths" of square feet, as indicated at the top of Table B.

Example 1. The sum of the end areas of two adjacent cross-sections is 93.0 sq ft. Find 93 in Table A. It is in the column headed 100 cu yd. The balance of the volume is found in Table B in the column headed 0.0 sq ft and opposite the figure 93 in Table A, or 72.2 cu yd. The total volume of earthwork in the 100-ft prism between the two adjacent cross-sections is therefore $100 + 72.2 = 172.2$ cu yd.

Example 2. The sum of the end areas of two adjacent cross-sections is 408.6 sq ft. Find 408 in Table A. It is in the column headed 700 cu yd. The balance of the volume is found in Table B in the column headed 0.6 sq ft and opposite the figure 408 in Table A, or 56.7 cu yd. The total volume of earthwork in the 100-ft prism between the two adjacent cross-sections is therefore $700 + 56.7 = 756.7$ cu yd.

These tables are based on cross-sections 100 ft apart; when adjacent cross-sections are not 100 ft apart, multiply the volume obtained from the tables by the distance between the cross-sections and divide by 100.

VOLUME OF EARTHWORK, IN CUBIC YARDS, FOR 100-FT PRISMS, CORRESPONDING

TABLE A

SQUARE FEET

1000 Cu Yd	900 Cu Yd	800 Cu Yd	700 Cu Yd	600 Cu Yd	500 Cu Yd	400 Cu Yd	300 Cu Yd	200 Cu Yd	100 Cu Yd	00 Cu Yd
540	486	432	378	324	270	216	162	108	54	0
541	487	433	379	325	271	217	163	109	55	1
542	488	434	380	326	272	218	164	110	56	2
543	489	435	381	327	273	219	165	111	57	3
544	490	436	382	328	274	220	166	112	58	4
545	491	437	383	329	275	221	167	113	59	5
546	492	438	384	330	276	222	168	114	60	6
547	493	439	385	331	277	223	169	115	61	7
548	494	440	386	332	278	224	170	116	62	8
549	495	441	387	333	279	225	171	117	63	9
550	496	442	388	334	280	226	172	118	64	10
551	497	443	389	335	281	227	173	119	65	11
552	498	444	390	336	282	228	174	120	66	12
553	499	445	391	337	283	229	175	121	67	13
554	500	446	392	338	284	230	176	122	68	14
555	501	447	393	339	285	231	177	123	69	15
556	502	448	394	340	286	232	178	124	70	16
557	503	449	395	341	287	233	179	125	71	17
558	504	450	396	342	288	234	180	126	72	18
559	505	451	397	343	289	235	181	127	73	19
560	506	452	398	344	290	236	182	128	74	20
561	507	453	399	345	291	237	183	129	75	21
562	508	454	400	346	292	238	184	130	76	22
563	509	455	401	347	293	239	185	131	77	23
564	510	456	402	348	294	240	186	132	78	24
565	511	457	403	349	295	241	187	133	79	25
566	512	458	404	350	296	242	188	134	80	26
567	513	459	405	351	297	243	189	135	81	27
568	514	460	406	352	298	244	190	136	82	28
569	515	461	407	353	299	245	191	137	83	29
570	516	462	408	354	300	246	192	138	84	30
571	517	463	409	355	301	247	193	139	85	31
572	518	464	410	356	302	248	194	140	86	32
573	519	465	411	357	303	249	195	141	87	33
574	520	466	412	358	304	250	196	142	88	34
575	521	467	413	359	305	251	197	143	89	35
576	522	468	414	360	306	252	198	144	90	36
577	523	469	415	361	307	253	199	145	91	37
578	524	470	416	362	308	254	200	146	92	38
579	525	471	417	363	309	255	201	147	93	39
580	526	472	418	364	310	256	202	148	94	40
581	527	473	419	365	311	257	203	149	95	41
582	528	474	420	366	312	258	204	150	96	42
583	529	475	421	367	313	259	205	151	97	43
584	530	476	422	368	314	260	206	152	98	44
585	531	477	423	369	315	261	207	153	99	45
586	532	478	424	370	316	262	208	154	100	46
587	533	479	425	371	317	263	209	155	101	47
588	534	480	426	372	318	264	210	156	102	48
589	535	481	427	373	319	265	211	157	103	49
590	536	482	428	374	320	266	212	158	104	50
591	537	483	429	375	321	267	213	159	105	51
592	538	484	430	376	322	268	214	160	106	52
593	539	485	431	377	323	269	215	161	107	53
1000	900	800	700	600	500	400	300	200	100	00

CUBIC YARDS FOR THE SUM

500 = 925 9
1000 = 1851 9
1500 = 2777.8
2000 = 3703 7
2500 = 4629.6

TO VARIOUS SUMS OF END-AREAS (IN SQUARE FEET) OF END CROSS-SECTIONS

TABLE B

0.0 Sq Ft	0.1 Sq Ft	0.2 Sq Ft	0.3 Sq Ft	0.4 Sq Ft	0.5 Sq Ft	0.6 Sq Ft	0.7 Sq Ft	0.8 Sq Ft	0.9 Sq Ft
Cubic Yards									
0.0	0.2	0.4	0.6	0.7	0.9	1.1	1.3	1.5	1.7
1.9	2.0	2.2	2.4	2.6	2.8	3.0	3.2	3.3	3.5
3.7	3.9	4.1	4.3	4.4	4.6	4.8	5.0	5.2	5.4
5.6	5.7	5.9	6.1	6.3	6.5	6.7	6.9	7.0	7.2
7.4	7.6	7.8	8.0	8.2	8.3	8.5	8.7	8.9	9.1
9.3	9.4	9.6	9.8	10.0	10.2	10.4	10.6	10.7	10.9
11.1	11.3	11.5	11.7	11.9	12.0	12.2	12.4	12.6	12.8
13.0	13.2	13.3	13.5	13.7	13.9	14.1	14.3	14.4	14.6
14.8	15.0	15.2	15.4	15.6	15.7	15.9	16.1	16.3	16.5
16.7	16.9	17.0	17.2	17.4	17.6	17.8	18.0	18.2	18.3
18.5	18.7	18.9	19.1	19.3	19.4	19.6	19.8	20.0	20.2
20.4	20.6	20.7	20.9	21.1	21.3	21.5	21.7	21.9	22.0
22.2	22.4	22.6	22.8	23.0	23.2	23.3	23.5	23.7	23.9
24.1	24.3	24.4	24.6	24.8	25.0	25.2	25.4	25.6	25.7
25.9	26.1	26.3	26.5	26.7	26.9	27.0	27.2	27.4	27.6
27.8	28.0	28.2	28.3	28.5	28.7	28.9	29.1	29.3	29.4
29.6	29.8	30.0	30.2	30.4	30.6	30.7	30.9	31.1	31.3
31.5	31.7	31.9	32.0	32.2	32.4	32.6	32.8	33.0	33.2
33.3	33.5	33.7	33.9	34.1	34.3	34.4	34.6	34.8	35.0
35.2	35.4	35.6	35.7	35.9	36.1	36.3	36.5	36.7	36.9
37.0	37.2	37.4	37.6	37.8	38.0	38.2	38.3	38.5	38.7
38.9	39.1	39.3	39.4	39.6	39.8	40.0	40.2	40.4	40.6
40.7	40.9	41.1	41.3	41.5	41.7	41.9	42.0	42.2	42.4
42.6	42.8	43.0	43.2	43.3	43.5	43.7	43.9	44.1	44.3
44.4	44.6	44.8	45.0	45.2	45.4	45.6	45.7	45.9	46.1
46.3	46.5	46.7	46.9	47.0	47.2	47.4	47.6	47.8	48.0
48.2	48.3	48.5	48.7	48.9	49.1	49.3	49.4	49.6	49.8
50.0	50.2	50.4	50.6	50.7	50.9	51.1	51.3	51.5	51.7
51.9	52.0	52.2	52.4	52.6	52.8	53.0	53.2	53.3	53.5
53.7	53.9	54.1	54.3	54.4	54.6	54.8	55.0	55.2	55.4
55.6	55.7	55.9	56.1	56.3	56.5	56.7	56.9	57.0	57.2
57.4	57.6	57.8	58.0	58.2	58.3	58.5	58.7	58.9	59.1
59.3	59.4	59.6	59.8	60.0	60.2	60.4	60.6	60.7	60.9
61.1	61.3	61.5	61.7	61.9	62.0	62.2	62.4	62.6	62.8
63.0	63.2	63.3	63.5	63.7	63.9	64.1	64.3	64.4	64.6
64.8	65.0	65.2	65.4	65.6	65.7	65.9	66.1	66.3	66.5
66.7	66.9	67.0	67.2	67.4	67.6	67.8	68.0	68.2	68.3
68.5	68.7	68.9	69.1	69.3	69.4	69.6	69.8	70.0	70.2
70.4	70.6	70.7	70.9	71.1	71.3	71.5	71.7	71.9	72.0
72.2	72.4	72.6	72.8	73.0	73.2	73.3	73.5	73.7	73.9
74.1	74.3	74.4	74.6	74.8	75.0	75.2	75.4	75.6	75.7
75.9	76.1	76.3	76.5	76.7	76.9	77.0	77.2	77.4	77.6
77.8	78.0	78.2	78.3	78.5	78.7	78.9	79.1	79.3	79.4
79.6	79.8	80.0	80.2	80.4	80.6	80.7	80.9	81.1	81.3
81.5	81.7	81.9	82.0	82.2	82.4	82.6	82.8	83.0	83.2
83.3	83.5	83.7	83.9	84.1	84.2	84.4	84.6	84.8	85.0
85.2	85.4	85.6	85.7	85.9	86.1	86.3	86.5	86.7	86.9
87.0	87.2	87.4	87.6	87.8	88.0	88.2	88.3	88.5	88.7
88.9	89.1	89.3	89.4	89.6	89.8	90.0	90.2	90.4	90.6
90.7	90.9	91.1	91.3	91.5	91.7	91.9	92.0	92.2	92.4
92.6	92.8	93.0	93.2	93.3	93.5	93.7	93.9	94.1	94.3
94.4	94.6	94.8	95.0	95.2	95.4	95.6	95.7	95.9	96.1
96.3	96.5	96.7	96.9	97.0	97.2	97.4	97.6	97.8	98.0
98.2	98.3	98.5	98.7	98.9	99.1	99.3	99.4	99.6	99.8
0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9

OF THE END AREAS OVER 500

3000 = 5555.6
3500 = 6481.5
4000 = 7407.4
4500 = 8333.3

APPENDIX C

PROBLEMS

CHAPTER I

1. Prepare a statement describing the administration of rural highway systems in your state. How are the officials selected who are in direct charge of each system? How many miles of highway are in each system? (The last annual report of the organization in charge of the state highway system should be consulted.)

CHAPTER II

1. Plot the grading, or mechanical analysis, curve of soil 8 in Table 1, and determine the effective size of the soil particles. Plot the percentage of particles smaller than a given size as ordinates and size of particles as abscissas. Semi-logarithmic coordinate paper will be found most satisfactory.

TABLE 1

SOIL TEST RESULTS

Soil No.	Mechanical Analysis					Constants				
	Coarse Sand	Fine Sand	Silt	Clay	Colloids	Liquid Limit	Plasticity Index	Shrinkage Limit	Moisture Equivalent	
	2.0 to 0.25 mm	0.25 to 0.05 mm	0.05 to 0.005 mm	Below 0.005 mm	Below 0.001 mm				Centrifuge	Field
1	54	27	12	7	5	22	2	18	11	19
2	20	78	1	1	0	20	0	0	3	21
3	0	7	55	38	24	51	29	17	36	39
4	0	11	57	32	16	55	36	13	55*	36
5	9	26	46	19	4	32	14	22	33	27
6	1	4	60	35	16	30	7	18	28	21
7	48	26	6	20	16	30	11	18	13	23
8	74	18	3	5	3	11	0		5	16
9	51	25	16	8	6	17	4	15	13	14
10	1	3	24	72	34	75	41	27		43

* Waterlogged.

2. Classify the various soils of Table 1 into the U. S. P. R. A. uniform subgrade groups.

3. Soil 5 occurred in Minnesota and heaved badly in the winter. If this soil occurred in thick layers of great length, what methods would you suggest for preventing frost heave?

4. A concrete pavement constructed on soil 3 cracked appreciably during the setting of the concrete. How would you treat a subgrade, composed of this soil, to prevent such cracking?

CHAPTER III

1. Determine the commercial size of concrete pipe required to drain 160 acres of flat farm land.

2. Calculate the required area of waterway opening for a culvert to drain 320 acres of steep and rocky land with abrupt slopes.

3. Calculate the rate of discharge in cubic feet per second by the Burkli-Ziegler formula for 40 acres of farm land whose average slope is 20 ft per 1000. The maximum rate of rainfall is 4 in. per hr. Determine the commercial size of circular clay pipe required for a length of 30 ft and a head of not over 0.4 ft on the pipe.

4. The inside edge of a straight concrete headwall for a 36-in. circular pipe culvert to be constructed across a road is to be on the outside shoulder line, and the top of the headwall is to be 4 in. above the outside shoulder line. The side slopes of the road embankment are 1½ : 1, and the headwall is 8 in. thick at the top. The bottom of the pipe is to be 4½ ft below the surface of the ground at the inside face of the headwall. The roadway on each side of the culvert will be in embankment. Determine the length of the headwall.

CHAPTER IV

1. Calculate the discharge of vehicles or the traffic capacity of a single traffic lane, for vehicle speeds from 10 to 60 mi per hr, by means of formulas 9, 10, and 11.

2. Two thousand vehicles passed through a vehicular tunnel on a single traffic lane in 1 hr at an average speed of 35 mi per hr. Calculate the average clearance between the vehicles.

3. A brick pavement to be constructed on a concrete base course has a width of 56 ft between curbs. What should be the minimum and maximum height of the center line of the pavement above the upper surface of the gutter adjacent to the curbs?

4. What should be the elevation of the top of a manhole located 17 ft from the center of a roadway 56 ft wide, if the crown is parabolic and equal at the center to ¼ in. per ft of half width? The elevation of the center of the roadway is 110.50 ft.

5. Design the cross-section of a residential street, 30 ft in width, on which it is desirable to use an unsymmetrical crown, the gutter on the high side being 1 ft higher than that on the low side. Six-inch curbs will be used, and the crown height, measured as for a symmetrical cross-section, will be determined in accordance with Moon's method, described in Appendix A.

6. A road on a sidehill location is on a 3° horizontal curve, and the distance, at a height of 5 ft, from the center of the roadway to the side slope of the

bank on the inside of the curve is 41 ft. What is the sight distance along the center line of the roadway? If a sight distance of 1000 ft is desired, how far back must the side bank be excavated?

7. Compute the total theoretical superelevation for a 20-ft paved roadway constructed on a curve, whose radius is 600 ft, for a vehicle speed of 30 mi per hr. Disregard friction.

8. Compute the total theoretical superelevation for an 18-ft paved roadway constructed on a curve whose degree of curvature is 6° for a vehicle speed of 40 mi per hr. Disregard friction.

9. Determine the amount of widening required for a paved roadway constructed on a curve whose radius is 600 ft.

10. Determine the amount of widening required for a paved roadway constructed on a curve whose degree of curvature is 6°.

11. Derive formula 18.

12. The point of intersection of two grade tangents occurs at Station 46 + 00 at an elevation of 52.40 ft. The back tangent has a grade of plus 5 per cent and the forward tangent a grade of minus 1 per cent. Calculate the elevations for each 50-ft station on a vertical parabolic curve 500 ft long.

13. A minus 4 per cent grade meets a minus 0.5 per cent grade at Station 124 + 50 at an elevation of 725.30 ft. Calculate the elevations for each 50-ft station on a vertical parabolic curve 300 ft long. What would be the elevation on the vertical curve for Station 123 + 87?

14. A grade tangent of plus 2 per cent has an elevation of 141.60 ft at Station 21 + 00, and a grade tangent of minus 3 per cent has an elevation of 139.60 at Station 22 + 00. Compute the station and elevation of the intersection of the grade tangents. Determine the elevation of the vertical curve at the point of intersection for a parabolic curve 600 ft long.

15. If Z is the distance from the beginning of a vertical curve to the low point on a vertical curve in a sag, prove that $Z = G_1L/(G_1 - G_2)$. An inlet is to be located at the low point of a gutter which is constructed on a vertical curve. The back tangent has a grade of minus 2.5 per cent, and the forward tangent a grade of plus 0.5 per cent. The intersection of the grade tangents is at Station 25 + 50 at elevation 656.50 ft. The vertical parabolic curve is 300 ft long. Determine the station and elevation of the gutter for the point where the inlet should be placed.

16. A north-and-south street intersects an east-and-west street. Design the intersection for the following data:

Widths:	
North-and-south street:	
Between property lines	50 ft
Between curbs	30 ft
East-and-west street:	
Between property lines	60 ft
Between curbs	40 ft
Grades:	
North-and-south street, going north:	
South of intersection	+6 per cent
North of intersection	+4 per cent
East-and-west street, going east:	
West of intersection	+6 per cent
East of intersection	+3 per cent

The established elevation of the center of the intersection is Elev. 100.

17. A north-and-south street intersects an east-and-west street. Design the intersection for the following data:

Widths:	
North-and-south street:	
Between property lines	50 ft
Between curbs	28 ft
East-and-west street:	
Between property lines	70 ft
Between curbs	46 ft
Grades:	
North-and-south street, going north:	
South of intersection	+4.8 per cent
North of intersection	+3.3 per cent
East-and-west street, going west	
West of intersection	+4.5 per cent
East of intersection	+3.6 per cent

The established elevation of the center of the intersection is Elev. 50.

CHAPTER V

1. Furnish each student a set of plans for improving a rural highway. Show on the plan the location and marking of the slope stakes for one cross-section in cut and one cross-section in fill.

2. Prepare a statement showing the type of grading machinery best adapted for the project of Problem 1.

3. Refer to *Proceedings, Highway Research Board,* Vol. 18 (1938), Part II, pp 157–160; list the factors which affect the value of the compacted unit weight of soil as determined by the Proctor test. What procedure, relative to the rigidity of the support upon which the compaction is performed, is followed by the National Park Service?

4. Consult the same reference as in Problem 3, pp. 158–174. Outline the procedure employed by the Ohio State Highway Department in determining the degree of compaction of embankments.

CHAPTER VI

1. One hundred pounds of Trinidad Lake asphalt whose specific gravity is 1.397 is fluxed with 12 lb of asphalt flux whose specific gravity is 1.007. Calculate the specific gravity of the fluxed asphalt.

2. Refined asphalt whose normal penetration is 25 points is to be fluxed so that the resulting asphalt cement will have a normal penetration of 65 points. If the addition of 1 lb of flux to 100 lb of refined asphalt increases the penetration of the asphalt 3 points, how many pounds of flux should be added to a 500-lb batch of refined asphalt?

3. Refined Trinidad asphalt has a bitumen content of 56 per cent. Sixty pounds of flux whose bitumen content is practically 100 per cent is added to 100 lb of refined Trinidad asphalt. Calculate the bitumen content of the resulting asphalt cement.

4. What would be the invoice volume of a tank car of asphalt cement at 60° F if the volume was 10,210 gal at a loading temperature of 340° F? The specific gravity of the asphalt cement is 1.03.

5. The purchase price of fluid tar is $0.05 per gal based on the volume of the tar at 60° F A tank car contains 9665 gal of fluid tar at the loading temperature of 285° F Calculate the cost of the tar

CHAPTER VII

1. Determine the thickness of a flexible pavement for a wheel load of 9000 lb on a single balloon tire, employing Goldbeck's procedure The bearing value of the subgrade was found from tests to be 30 lb per sq in

2. A truck equipped with dual tires on the rear wheels carries the legal maximum axle load of 18,000 lb What size of balloon tire is required? Assume that one of the tires carries the entire wheel load Determine the thickness of a flexible pavement to carry the load for a subgrade supporting value of 30 lb per sq in

3. Assume that the tires of Problem 2 are inflated to the same pressure as in Problem 1 Determine the thickness of flexible pavement

4. Assume that the tire of Problem 1 carries an overload of 100 per cent Determine the thickness of flexible pavement

CHAPTER VIII

1. A glacial gravel deposit, containing gravel of the composition shown below, is available for gravel road construction on a clay subgrade

	PER CENT
Passing 2-in screen, retained on 1-in screen	19
Passing 1-in screen, retained on ¼-in screen	40
Passing ¼-in screen, retained on 10-mesh sieve	8
Passing 10-mesh sieve, retained on 60-mesh sieve	14
Passing 60-mesh sieve retained on 200-mesh sieve	7
Passing 200-mesh sieve	12

The clay in this gravel is present in the form of a finely divided coating on the particles of stone Write a specification for the composition of the gravel surface which will permit the use of this material and produce a satisfactory roadway Describe how the gravel in the deposit must be treated to meet the above specifications

2. The materials described in Table 2 are available for the construction of a gravel type of soil road surface Determine the weight proportions of the materials which will give a satisfactory mixture How would you determine whether the binder in the mixture was satisfactory?

TABLE 2

TESTS OF BANK GRAVEL AND BINDER SOIL

	Bank, Gravel per cent	Binder Soil, per cent
Passing 1-in sieve	100	
Passing ¾-in sieve	92	
Passing ⅜-in sieve	70	
Passing No 4 sieve	41	100
Passing No 10 sieve	32	98
Passing No 40 sieve	17	90
Passing No 200 sieve	2	86
Plasticity index		24

3. The gravel and binder soil of Problem 2 weigh 95 and 75 lb per cu ft, respectively, at the source of supply. Determine the volume proportions in which to mix the two materials.

4. Assume that the road surface of Problem 1 will have the shape of cross-section shown in Fig. 58(d), and that the compacted mixture weighs 150 lb. per cu ft. How many cubic yards, loose measure, of each material will be required for 1 mile of road?

CHAPTER IX

1. Draw a cross-section of a waterbound macadam roadway (which will have a bituminous surface treatment) for two-lane traffic, and estimate the quantity of broken stone required for 1 mi of road.

2. Prepare an outline showing the number of men and the items of equipment required to construct a waterbound macadam roadway.

3. Estimate the quantity of materials required per mile of bituminous macadam pavement, 20 ft wide, for the cross-section shown in Fig. 65. Assume that the foundation course is of gravel and has a compacted thickness of 5 in. The estimate shall include the foundation course, the gravel shoulders, the broken stone base course, and the bituminous macadam layer. Disregard the bituminous surface treatment on the gravel shoulders.

CHAPTER X

1. Estimate the quantities of materials required for a bituminous carpet coat on the gravel shoulders of Problem 3, Chapter IX Assume the gravel shoulders are firmly bound Grade MC-1 liquid asphalt will be employed for the prime coat, and 85–100 penetration asphalt for the carpet coat. The stone chips for the cover coat shall vary from ½ in. to a No 4 sieve opening in size. Write a specification for the size and grading of the stone chips. (Consult the *A.S.T.M. Specifications for Standard Sizes of Coarse Aggregate for Highway Construction*)

2. Determine the percentage of road oil required by the modified McKesson-Frickstad formula for a bituminous surface composed of crushed gravel treated by the road-mix method. The grading of the aggregate is as follows:

	Per Cent
Passing 1-in., retained on 10-mesh	54.2
Passing 10-mesh, retained on 200-mesh	34.8
Passing 200-mesh	11.0

3. A bituminous surface 2.5 in. thick and 20 ft wide is to be constructed of graded mineral aggregate by the road-mix method. The compacted mineral aggregate has been found to weigh 135 lb per cu ft. The aggregate in a loose condition weighs 112 lb per cu ft. Calculate the number of cubic feet of loose mineral aggregate required per linear foot of roadway.

CHAPTER XI

1. Compute the distance in feet between transverse joints for no intermediate cracking for an 8-in. plain concrete pavement, 10 ft wide between longitudinal joints, for an allowable tensile strength of concrete of 30 lb per sq in. The

coefficient of subgrade friction is 1.5. Assume that concrete weighs 144 lb per cu ft.

2. A plain concrete beam, 6 in. by 6 in. in cross-section, was tested with center loading on a 24-in. span. The breaking load was 4250 lb. Determine the modulus of rupture which is the apparent breaking tensile stress in the outer fiber of the concrete.

3. Calculate the modulus of rupture for a plain concrete beam, 8 in. by 8 in. in cross-section, tested with loading at the third points on a 30-in. span. The breaking load registered on the testing machine was 5120 lb.

4. What is the effect of the weight of the beam of Problem 3 on the value of the modulus of rupture?

5. The maximum legal wheel load is 8000 lb on high-pressure tires. Consider that the load is applied at the edge of a plain concrete slab at a free transverse joint. Assume an allowable unit stress in flexure equal to 400 lb per sq in. Calculate the thickness of slab required, employing Older's procedure.

6. Determine, by Older's procedure, the thickness of a reinforced-concrete slab with dowels across longitudinal and transverse joints for a maximum wheel load of 8000 lb on high-pressure tires, assuming an allowable unit stress in flexure of 300 lb per sq in. Assume the cross-section of the slab has the shape shown in Fig. 88(*b*).

7. Plot the impact factors of Table 20 (vertically) against the static wheel loads, and determine, by projecting the diagrams, the total impact reaction for a static wheel load of 12,000 lb on dual high-pressure tires and for a static wheel load of 13,500 lb on dual balloon tires.

8. Determine, by the Westergaard procedure, the thickness of a concrete pavement carrying a wheel load of 8000 lb on dual high-pressure tires for a uniform unit stress (caused by load and temperature) throughout the cross-section of 400 lb per sq in. The length of the slab between transverse joints is 10 ft.

CHAPTER XII

1. Prepare an outline showing the inspectors required to supervise the construction of a concrete pavement on which a central proportioning plant and paving mixer are used, and list the duties of each inspector.

2. A concrete pavement has a width of 20 ft, an edge thickness of 9 in., and a center thickness of 6½ in. The edge thickness tapers uniformly to the center thickness in a distance of 24 in. The specifications require a minimum cement content of 1.50 barrels of cement per cubic yard of concrete. Determine the quantity of cement required per linear foot of pavement.

3. Draw a cross-section of an integral curb for a concrete pavement to be placed on a city street.

4. Prepare an estimate of the quantities of materials for 1 mi of reinforced-concrete pavement from a set of plans for a specific project.

CHAPTER XIII

1. Two sands, represented by the following mechanical analyses, are available for use in a sheet asphalt mixture. Determine the proportions of each sand in a mixture of the two sands which will produce a satisfactory grading.

	Per Cent	
	Sand *A*	Sand *B*
Retained 10-mesh	1	
Passing 10-mesh, retained on 20-mesh	8	
Passing 20-mesh, retained on 30-mesh	16	
Passing 30-mesh, retained on 40-mesh	23	
Passing 40-mesh, retained on 50-mesh	28	1
Passing 50-mesh, retained on 80-mesh	19	13
Passing 80-mesh, retained on 100-mesh	4	32
Passing 100-mesh, retained on 200-mesh	1	47
Passing 200-mesh	0	7

2. Two sands, represented by the following mechanical analyses, are available for use in a sheet asphalt mixture. Determine the proportions of each sand in a mixture of the two sands which will produce a satisfactory grading for heavy traffic conditions.

	Per Cent	
	Sand *A*	Sand *B*
Passing 10-mesh, retained on 20-mesh	0	23
Passing 20-mesh, retained on 30-mesh	0	34
Passing 30-mesh, retained on 40-mesh	4	22
Passing 40-mesh, retained on 50-mesh	11	16
Passing 50-mesh, retained on 80-mesh	40	5
Passing 80-mesh, retained on 100-mesh	21	0
Passing 100-mesh, retained on 200-mesh	23	0
Passing 200-mesh	1	0

3. Using the sand mixture obtained in Problem 2, design a sheet asphalt mixture to have 11 per cent bitumen and 14 per cent mineral filler finer than 200-mesh. The asphalt cement has a bitumen content of 99.5 per cent. The limestone dust to be used for filler has this grading:

	Per Cent
Passing 40-mesh, retained on 50-mesh	1
Passing 50-mesh, retained on 80-mesh	3
Passing 80-mesh, retained on 100-mesh	5
Passing 100-mesh, retained on 200-mesh	16
Passing 200-mesh	75

Show the sieve analysis of the sheet asphalt mixture. Check the grading of the sand in the sheet asphalt mixture to see that it approximates the heavy traffic standard sand grading.

4. The specifications for a sheet asphalt wearing course require 10.5 per cent bitumen, 20 per cent mineral filler, and 69.5 per cent sand by weight. The asphalt is Bermudez asphalt cement containing 95 per cent bitumen and 5 per cent mineral matter. The stone dust used for mineral filler has 80 per cent passing the 200-mesh sieve. The sand has 4 per cent passing the 200-mesh sieve. Compute the amounts of materials required for 100 lb of mixture.

5. The specifications for a sheet asphalt wearing course required 10.5 per cent bitumen, 15 per cent mineral filler, and 74.5 per cent sand by weight. The asphalt used was practically pure bitumen. The limestone dust used as mineral filler had 21.6 per cent retained on the 200-mesh sieve. Potomac River sand which was used had 9.1 per cent passing the 200-mesh sieve and 100 per cent passing the 10-mesh sieve. Compute the percentage of materials for the mixture.

6. The following data were obtained by the Le Chatelier flask method for determining the specific gravity of the limestone dust and Potomac River sand used in the mixture given in Problem 5.

MATERIAL	INITIAL READING WATER, CC.	FINAL READING WATER AND MATERIAL, CC.	WEIGHT OF MATERIAL, GM.
Limestone dust	0.40	18.57	50.0
Potomac River sand	0.80	19.65	50.0

The specific gravity of the asphalt was determined to be 1.013. Calculate the maximum theoretical density of the sheet asphalt mixture.

7. A sample of sheet asphalt mixture referred to in Problems 5 and 6 was compressed at a temperature of 250° F into the form of a cylindrical briquette, weighing 110.34 grams in air and 60.08 grams in water. Determine the percentage of voids in the mixture.

8. The proportions, by weight, and the specific gravities of each of the constituents of a sheet asphalt mixture are as follows:

	SPECIFIC GRAVITY	PER CENT BY WEIGHT
Sand	2.64	72
Limestone dust	2.76	18
Asphalt cement	1.048	10

The mixture, as compacted by the rollers, had a specific gravity of 2.27; the specific gravity of a sample compressed in the laboratory was 2.29; determine the maximum theoretical density and compare the efficiency of the rolling operation with the density obtained in the laboratory.

9. The proportions, by weight, and the specific gravities of each of the constituents of a sheet asphalt mixture are as follows:

	SPECIFIC GRAVITY	PER CENT BY WEIGHT
Sand	2.65	70
Limestone dust	2.70	19
Asphalt cement	1.02	11

The mixture, as compacted by the rollers on the street, had a specific gravity of 2.22. (*a*) Determine the maximum theoretical density. (*b*) Determine the percentage of voids. (*c*) Compute the weight of a square yard of 1½-in. wearing surface.

10. The proportions, by weight, and the specific gravities of each of the constituents of an asphaltic concrete mixture are as follows:

	SPECIFIC GRAVITY	PER CENT BY WEIGHT
Stone	2.70	50
Sand	2.65	39
Limestone dust	2.70	5
Asphalt cement	1.01	6

The mixture, as compacted by the rollers on the street, had a specific gravity of 2.36. (*a*) Determine the maximum theoretical density. (*b*) Determine the percentage of voids. (*c*) Compute the weight of a square yard of 2-in. asphaltic concrete surface. (*d*) If the specific gravity of a sample compressed in the laboratory was 2.40, compare the efficiency of the rolling operation with the density obtained in the laboratory.

11. A mixing plant has a capacity of 1500 sq yd of 2-in. asphaltic concrete wearing course per working day of 10 hours. Refer to Problem 10 for the proportions of the mixture. The capacity of the mixer is 1200 lb of mixture. The asphalt cement is practically 100 per cent bitumen. The sand and stone weigh 105 lb per cu ft. The limestone dust is not passed through the drier. (*a*) Calculate the required daily capacity of the drier and the melting tanks. (*b*) Determine the number of batches required to produce the 1500 sq yd of surfacing. (*c*) Compute the allowable time for charging the mixer if it requires 1 min to mix each batch thoroughly.

12. If Trinidad asphalt cement which is 76.7 per cent soluble in carbon disulphide is substituted for the asphalt cement given in Problem 11, calculate the required daily capacity of the melting tanks.

13. The percentages by weight of materials used in an asphaltic concrete mixture are asphalt cement 7 per cent, hydrated lime for filler 5 per cent, sand 33 per cent, and stone 55 per cent. The asphalt cement, hydrated lime, and stone are imported by railroad to a point of delivery 2.5 mi away from the end of a road project 1 mi long, making an average haul to the job of 3 mi. The sand is obtained from a pit 1.5 mi from the end of the job, making an average haul of 2 mi to the job. The sand pit is 3 mi from the railroad delivery. Assume the cost of hauling either raw materials or finished mixture as 20 cents per ton-mile. Two locations are available for the asphalt paving plant: (1) at the railroad delivery, and (2) at the sand pit. Where should the plant be located from the standpoint of the most economical haul? Arrange work in tabular form.

14. Prepare an outline showing the inspectors required for the supervision of the construction of a sheet asphalt pavement and the duties of each.

CHAPTER XIV

1. A Kentucky rock asphalt wearing course is to be placed on a new water-bound macadam foundation 21 ft in width. The rock asphalt layer is to be 20 ft in width and, after compaction by rolling, 1½ in. in thickness. A prime coat of ⅓ gal of Grade MC-2 liquid asphalt per square yard is to be applied and traffic allowed on the foundation. Immediately before the placing of the rock asphalt a tack coat of ⅛ gal of Grade RC-1 liquid asphalt per square yard is to be applied. Assume that the rock asphalt, after compaction by rolling, weighs 90 lb per sq yd per in. of thickness. Calculate the quantities of materials per mile of pavement exclusive of the foundation. Show the quantities of liquid asphalts in gallons and the quantity of rock asphalt in tons.

CHAPTER XV

1. Prepare an estimate showing the quantity of each of the following materials necessary for the construction of a mile of 18-ft brick pavement:

Sand for bituminous mastic.
Asphalt for bituminous mastic.
Paving brick, 2½ by 4 by 8½ in.
Asphalt filler.

Assume that the compacted mastic weighs 92 lb per sq yd per in. of thickness; also that the bricks have side lugs and end lugs, each ⅛ in. in height.

CHAPTER XVI

1. A county road system embraces the following types of road surfaces:

Brick on concrete base course	2 mi
Coarse-graded asphaltic concrete:	
On concrete base course	5 mi
On waterbound macadam base course	28 mi
Concrete	8 mi
Bituminous macadam, asphalt binder:	
18 ft wide	8 mi
20 ft wide	10 mi
Waterbound macadam:	
10 ft wide	100 mi
16 ft wide	81 mi
18 ft wide	260 mi
Gravel, untreated, not stabilized	294 mi
Unimproved earth	522 mi

Prepare a list of the operations involved in the maintenance of each surface, and indicate the kind of equipment which you think should be provided for these operations.

2. Assume that the macadam road surfaces of Problem 1 require a bituminous surface treatment every 8 years and that a carpet coat, ½ in. in thickness, will be satisfactory. Assume the following average prices:

Cost of stone, placed on the road surface, per ton	$2.20
Cost of cut-back asphalt, Grade RC-3, placed on the road surface, per gallon	0.11
Cost of rolling, per mile	12.00

What amount of money, on the average, must be included in the annual budget for surface treatment?

3. Prepare a statement showing the responsibility of public highway departments in your state for the condition of highways. Does the common-law rule of "no liability" apply, or is the responsibility fixed by statute?

CHAPTER XVII

1. How would you arrange for convenient access to governmental survey corners on a section line road on which the section line falls within the paved area?

2. If a set of drawings showing the topographical details of a section of rough country can be conveniently obtained, select tentative locations for a highway between fixed points.

3. Inspect several railroad and highway intersections in or near your city and suggest one crossing for which grade separation would be especially desirable. What type of separation would you select? Give the reasons for your answer. How would the structure affect the value of adjacent property?

4. Assume that an unimproved land-service road, on which the natural ground surface has gentle slopes with good drainage facilities, is to be improved with a gravel or other low-cost surface. All grading operations can be performed with a blade grader. Assume that the road runs at an angle with the governmental land lines and has several curves; the boundaries of the road are not well defined, and no previous survey records are available. Prepare a statement showing how you would proceed with a survey and plans for improv-

ing the road by day labor. How would you set stakes for the grading and surfacing?

5. Give the class the plans for a section of rural highway which embraces a large amount of earthwork on which the grade has been designed. Determine the quantity of overhaul.

6. Prepare an estimate of quantities for a section of rural highway, using a set of plans which have been prepared.

CHAPTER XVIII

1. A gravel road surface is constructed to a width of 20 ft and to a compacted depth of 8 in., at a cost of $0.55 per sq yd The average daily traffic is 250 vehicles. Calcium chloride is used as a dust layer, and three applications of 1, 0.75, and 0.75 lb per sq yd to a width of 18 ft are required annually; the cost of applying the calcium chloride, including the cost of the material, is $27 per ton. Assume that the annual cost of floating the surface is $75 per mile and of patching is $60 per mile. Assume the annual cost of periodic maintenance to be $250 per mile. Calculate the annual cost of the wearing surface, exclusive of administration. Assume an interest rate of 3 per cent

2. Assume that the gravel road in Problem 1 may be used as the base for a low-cost bituminous wearing course 2½ in. in thickness and 19 ft in width. Assume that the bituminous wearing surface may be constructed at a cost of $0.35 per sq. yd. Assume the annual cost of ordinary maintenance to be as follows:

Patching surface	$60 per mi

Assume the annual cost of periodic maintenance to be as follows:

Minor replacements	$25 per mi
Major replacements	$150 per mi

Calculate the annual cost of operation of the highway, exclusive of administration.

3. A bituminous macadam pavement, 18 ft wide, has an initial cost of $1.00 per sq yd, an annual maintenance cost of $250 per mi, and a periodic maintenance cost of $290 per yr per mi. A concrete pavement of the same width has an initial cost of $1.70 per sq yd, an annual maintenance cost of $150 per mi, and a periodic maintenance cost of $800 per yr per mi. The costs of administration are included in the maintenance values. The rate of interest may be taken as 4 per cent. Assume that the cost of vehicle operation on the concrete pavement is ¼ cent per vehicle-mile less than on the bituminous macadam pavement. Which type is the more economical for the conditions stated for an average daily traffic of 1000 vehicles?

4. A concrete pavement cost $25,000 per mi initially; it has an annual maintenance cost of $140 per mi and a periodic maintenance cost of $750 per yr per mi. A gravel road cost $3000 per mi initially; it has an annual maintenance cost of $400 per mi and a periodic maintenance cost of $500 per yr per mi. The costs of administration are included in the maintenance values. The rate of interest may be taken as 4 per cent. The cost of vehicle operation is 5 cents per vehicle-mile on concrete and 5¾ cents on gravel. What is the saving in annual transportation cost effected by changing from a gravel surface to a concrete pavement for an average traffic of 500 vehicles per day?

5. A city *A* is located 20 mi from a point *B*. The bearing of line *BA* is N—45° E. A city *C* is located 20 mi south of point *B*. Two highway routes are under consideration between the two cities: (1) the route *ABC*, and (2) the straight line between *A* and *C*. The annual cost of the route *ABC* is estimated to be $1500 per mi and of the route *AC* $1800 per mi. The average daily traffic between *A* and *C* is 1200 vehicles per day. The variable cost of vehicle operation is estimated to be $0.04 per vehicle-mile. The fixed cost of vehicle operation may be considered to be the same for both locations. Which route is the more economical?

6. A city *A* is located 10 mi east of a city *B*, and another city *C* is located 10 mi south of *B*. Two road systems connecting these cities are under consideration: (1) that running directly from *A* to *B* and *B* to *C*, and (2) that running directly from *A* to *C* with a branch road from *D*, the middle point of line *AC*, to *B*. All roads are to be of the concrete type with an estimated annual cost of $2000 per mi except *BD* which is to be constructed with a gravel surface having an estimated annual road cost of $500 per mi. The variable cost of vehicle operation is estimated to be $0.04 per vehicle-mile over all roads. The fixed cost of vehicle operation may be considered to be the same for both locations. The average number of vehicles per day traveling from *A* to *C* is 1000, from *A* to *B* 100, and from *B* to *C* 200. Determine which of the two road systems would be the more economical.

7. An automobile which weighs 2500 lb and has a frontal area of 20 sq ft has a maximum tractive force available at the wheels in high gear of 270 lb at a speed of 30 mi per hr. Determine the percentage of grade that the car can pull at this speed on a gravel road having a rolling resistance of 45 lb per ton.

CHAPTER XIX

1. Make an outline map of your city showing the location of the principal traffic thoroughfares and the connections with the principal rural highways in the adjacent territory.

2. Figure 1 represents a portion of a county in which a traffic survey is to be conducted to obtain accurate information relative to traffic density on all roads. Reproduce the sketch, and show by means of appropriate symbols where you would locate primary and secondary observation stations to determine the traffic density on all roads. Prepare a schedule for a traffic survey and an estimate of the cost of doing the work. The maximum traffic and the average daily traffic are desired.

3. A county road organization is planning on the issue of bonds to defer the payment of the cost of improvement of a large portion of a county-wide highway system. The bond issue will involve a total of $5,000,000, and payments on the principal are to be deferred until the end of 6 years, the bonds then to be retired in 10 years at a uniform sum yearly. Assume interest at 5 per cent. Calculate the yearly payments each year for the entire period the bonds are outstanding.

4. Assume, for the conditions stated in Problem 3, that the county has an assessed valuation of $337,423,000. What will be the tax rate for each year, per $1000 of property, required to pay the interest and retire the bonds?

5. A township issued road bonds of the serial type to the amount of $100,000 for a 10-yr period at 4 per cent interest; $10,000 of the principal matured annually. Calculate the interest and tax for each year.

6. Solve Problem 5 assuming that 4 per cent sinking-fund bonds were issued instead of serial bonds. The sinking fund was not invested.

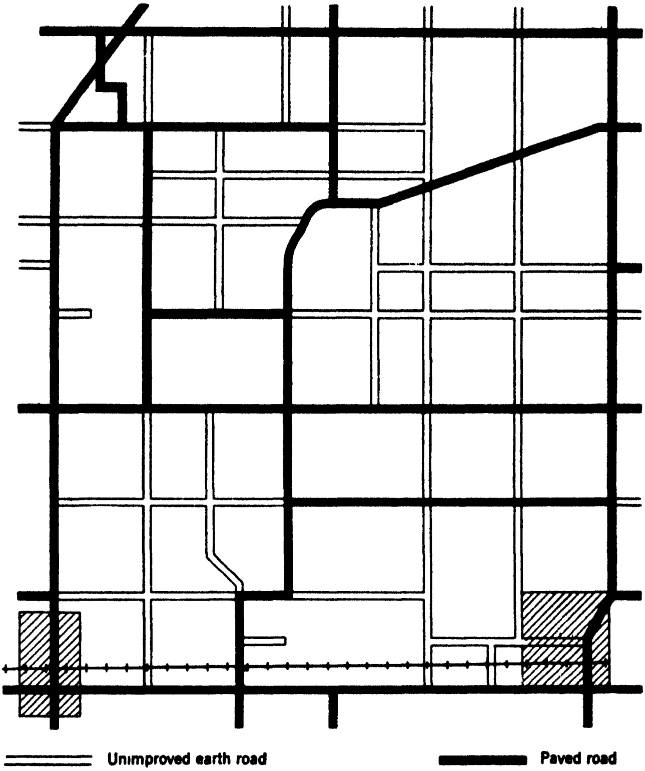

FIG. 1.

7. Calculate the amount of the annual deposit to the sinking fund, invested at 4 per cent compound interest, to yield $100,000 in 10 yr (*Suggestion:* Use formula $A = \frac{Pr}{(1 + r)^N - 1}$, a discussion of which can be found in *U.S.D.A. Bulletin* 136 by Hewes and Glover.)

8. Calculate the annual interest and tax for Problem 7 for each year, assuming that the sinking fund was invested at 4 per cent compound interest. Are serial bonds at 4 per cent more economical than this method?

9. A system of roads 100 mi long was constructed at an average cost of $25,000 per mi. The funds were obtained from the sale of 4 per cent serial bonds divided into 20 equal series, the first series maturing in one year and the last series in 20 years. (*a*) What was the total tax levy required? (*b*) If the pay-as-you-go plan of financing had been used, what percentage greater mileage could have been constructed from the tax levy required for the bonding method? (*c*) How many year-miles of service are obtained under each method? Assume the time required for construction as 1 year. (*d*) Discuss the advantages and disadvantages of the two systems of financing.

10. The pavement on Maple Avenue shown in Fig. 2 is 36 ft wide and cost $2.25 per sq yd. One-half of this cost is to be assessed against block *A*. Also on Maple Avenue abutting on block *A* are curb and gutter which cost $0.75 per ft, and concrete sidewalk, 5 ft wide, which cost $0.15 per sq ft. The cost of the improvement of the street intersections is not to be considered. Calculate the total assessment for these improvements for each lot in block *A* according to the frontage method.

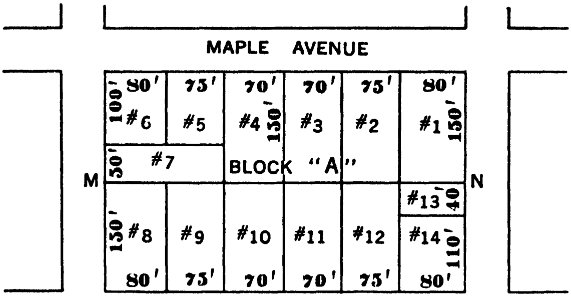

FIG. 2.

11. Referring to Problem 10, calculate the assessment for each lot in block *A* by the area distance method for a maximum assessed distance of 150 ft from the improvement (*Suggestion:* Take moments of areas about the axis *MN*.)

12. Solve Problem 11 for a maximum assessed distance of 300 ft from the improvement.

13. Assume that the average highway vehicle uses the highways to the amount of 12,000 mi annually, and that 15 mi of travel are obtained per gallon of gasoline. The average gasoline tax is $0.04 per gal, and the average license fee is $10.00. Calculate the average annual tax payment per vehicle per mile, and determine the average traffic income per year per mile of highway derived from gasoline taxes and license fees for traffic volumes of 250, 500, 1000, 1500, and 2000 vehicles per day.

14. The initial investment for a cold-laid plant-mix bituminous surface is $7000 per mi; that for an untreated gravel road is $3000. Assume the interest rate at 4 per cent, annual maintenance cost at $250 per mi, and periodic maintenance cost per year at 3½ per cent of the initial investment for both types of surfaces. Assume that the annual cost of highway operation should be no greater than the annual payments made by road users. Assuming the conditions stated in Problem 13, which type of surface should be selected for a highway which has an average traffic volume of 500 vehicles per day?

15. A highway paved with a concrete roadway cost $30,000 per mi initially, including the cost of structures, grading, and drainage. Assume an annual maintenance cost of $125 per mi, and an annual deposit of $500 per mi to provide for periodic maintenance. Assume that the cost of administration is included in the maintenance values. Referring to the conditions stated in Problem 13, determine the number of vehicles per day required to yield sufficient annual payments in taxes for the use of the highway to equal the annual cost of highway operation. Take the rate of interest at 4 per cent.

16. A four-lane concrete highway has an average traffic volume of 2500 vehicles per day. The annual cost of highway operation is $3300 per mi. Determine the amount of surplus of traffic income, assuming that the average vehicle uses the highways to the amount of 7500 mi annually, travels 15 mi on each gallon of gasoline, and pays a gasoline tax of $0.05 per gal and a license fee of $15.

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